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STRUCTURAL EVALUATION AND DESIGN OF BLOCK PAVEMENTS FOR CONTAINERS TERMINALS

by

J.H. KEYSER, Ph.D., eng.

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Centre de recherche et de contrôle appliqué à la construction (C.R.C.A.C. Inc.)

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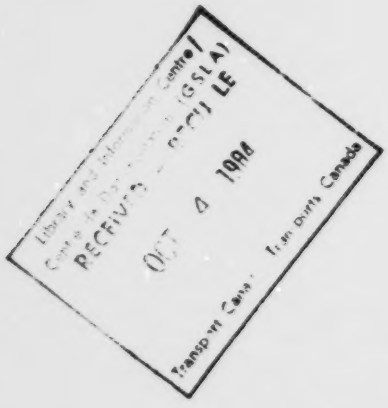

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17. Abstract

The objective of this study is to examine the comparative economic and performance advantages of using concrete paving blocks in lieu of asphalt paving for heavy duty applications at marine terminals. The report is divided in two parts:

a) The structural and functional description of concrete block pavement and existing pavement design methods, and (b) A field evaluation of the performance of experimental concrete blocks pavements and their comparison with a normalized asphalt pavement.

Six sections of test pavement have been built in the Port of Montreal to evaluate, under canadian conditions, the effects of the following factors on pavement performance: pavement types (block VS asphalt); type of base under concrete blocks (cement-treated VS granular); and block thickness (80 mm VS 100 mm). A description of the design, the construction and the economic analysis of each pavement section is given in the report as well as a description and the results of a monitoring program which has been designed to assess the condition of pavement component layers and soil, and the degradation of the pavement with time. Results obtained are also given in the report: the change in structural characteristics during construction; the effects of compaction of block pavement with a 30 t rubber-type roller; the change in thermal and moisture conditions; the change in structural behavior and the degradation of pavement.

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17. Résumé

Le but spécifique de l'étude est de comparer les avantages et désavantages des chaussées de pavés avec ceux de la chaussée flexible conventionnelle et ce tant sur le plan économique que celui du comportement en service. Ainsi le rapport est divisé en deux parties:

a) de la description structurale et fonctionnelle des chaussées de pavés et des méthodes de conceptions existantes et (b) d'une étude expérimentale comparative entre les chaussées en pavés et la chaussée flexible normalisée.

Six chaussées d'essai ont été construites au Port de Montréal dans le but d'évaluer l'effet des facteurs suivants sur le comportement des chaussées sous le climat canadien: Le type de chaussée (pavés versus revêtement bitumineux), type de base sous les chaussées en pavé (pierre concassée stabilisée ou non au ciment) et épaisseur des pavés (80 mm VS 100 mm). Dans le rapport, l'auteur décrit les méthodes de conception et de construction utilisées, les résultats de l'analyse économique, ainsi que les méthodes d'évaluation utilisées pour suivre l'évolution des caractéristiques des couches de la chaussée et du sol, enfin que la dégradation de la chaussée dans le temps. On trouvera également les résultats d'évaluation: l'évolution de la capacité structurale durant la construction; l'effet du compactage avec un rouleau pneumatique de 30 tonnes; les changements de condition de température et d'humidité dans les couches de la chaussée; les changements du comportement structural des chaussées et leurs dégradations.

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Chaussée en pavés, chaussée flexible, conception, évaluation, chaussée d'essai, régime thermique, régime hydraulique, comportement, dégradation	Nombre limité de copies disponible au Centre de développement des transports

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1. INTRODUCTION

History and development

Although pavements constructed of shaped blocks date from the Middle Ages, they were not in common use until the 20th Century. These earliest paving elements were rectangular blocks of stone or wood and towards the end of the 1800's clay bricks also appeared. The viability of the latter was secured with the advent of rubber tires.

It was not until the end of the second world war that the use of concrete paving blocks became widespread. This was especially so in Holland in the early fifties where clay bricks were already used because of the scarcity of exploitable quarries. These blocks were for the most part rectangular with less than ten percent of the units shaped in the form of an interlocking-I. Prior to this, preference had been given to the rectangular shaped paving block not only because it was the traditional shape of customary practice but also because of the ability of this block pavement to adapt to infrastructural mouvements that might be as great as one metre. Successful experience with concrete block pavements in Holland led to a significant increase in production facilities there and elsewhere. In 1956, 25 plants produced a total of 1,5 million square metres of paving blocks and four short years later 40 plants were producing some five million square metres of concrete paving blocks while the production of clay units was in the order of six to seven million square metres.

In Germany concrete paving blocks had been in use since the end of the 19th Century, but it was not until 1936 that construction of the first block pavement test road for heavy traffic was undertaken. Although the blocks used at that time were rectangular (measuring 240 mm x 120 mm x 80 mm), unlike Holland, they had already been producing interlocking as well as other types of paving blocks. Today, more than 200 types are marketed in Germany.

Between 1970 and 1980, more than 250 million square metres of paving block roads were built in Germany and the annual production of 1980 was in the order of 44 million square metres. Of this production 36% was used for roads, 30% for industrial floors, 12% for pedestrian walks, 7% for parks, schools and service stations with the remaining 15% directed to domestic needs and other secondary applications.

The success of concrete block pavements in Germany and Holland spread rapidly to Belgium, Austria and Denmark and in the early seventies they crossed the Channel to the United

Kingdom. Continental Europe's utilization of concrete paving blocks is now estimated at 0.5 - 1.0 m^2 per person.

Not until the 60's did concrete paving blocks make their appearance in the Americas: at first in Brazil and Argentina and then, during the 70's, in Canada and the U.S.A. where one million square metres of block were installed in 1980. Finally, South Africa and Australia were included among the world's largest producers with a 1980 yearly output of 7,3 and 3,0 million square metres of paving blocks, respectively.

2. STRUCTURAL AND FUNCTIONAL DESCRIPTION OF CONCRETE BLOCK PAVEMENT

2.1 Elements Comprising a Concrete Block Pavement

Concrete block pavements are generally classified as flexible or semi-rigid (see figure 2.1)and consist of three distinct structural zones: a surface layer consisting of blocks and a bedding sand; one or more bearing layers (base and foundation); and finally, a graded and compacted layer of soil (subgrade).

2.1.1 Surface Layer

The two components of this layer, the paving stone and the bedding sand, serve to assure both a closely-knit and durable surface as well as structural solidity and flexibility.

2.1.1.1 Concrete Paving Blocks

Paving block units are classified as either unlocked or interlocking. The unlocked type is characterized by a geometrical shape where the smooth sides form only obtuse angles that permit a relative sliding movement of the blocks. On the other hand, the sides of the interlocking block are angled or curved concavely.

The geometric characteristics of this latter type therefore act to prevent the sliding movement mentioned above. Figure 2.2, from Shackel[1], illustrates the different geometric forms of paving block in use. According to Bergerhof[2] more than 40 geometric shapes of paving blocks have been patented in Europe. One notes in Figure 2.2 that certain interlocking blocks have that feature at the transverse ends while others are keyed only along their longitudinal faces. In general the interlocking of the block is greater in the first case since the block is tied in throughout the perimeter length. Certain paving blocks have their upper edges bevelled. This feature results in:

- An improved aesthetic appearance, where a play of shadows hides profile irregularities;

- Facilitating the laying of the block;

- Reducing the risk of chipping edges when there is vertical rotation of the blocks; and

- Reducing the tendancy of the bedding sand in the joints to be removed by tire traffic.

FIGURE 2.1

TYPICAL SECTION OF A BLOCK PAVEMENT

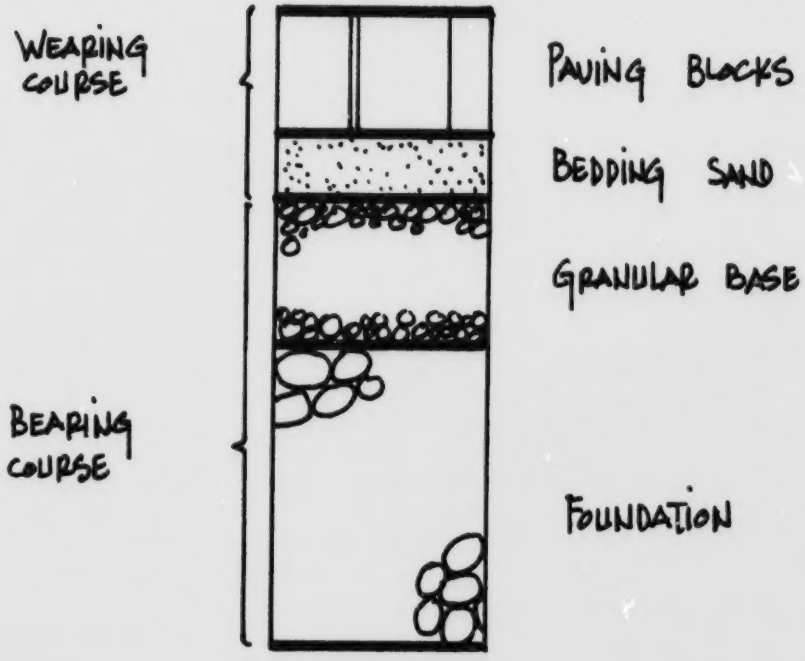
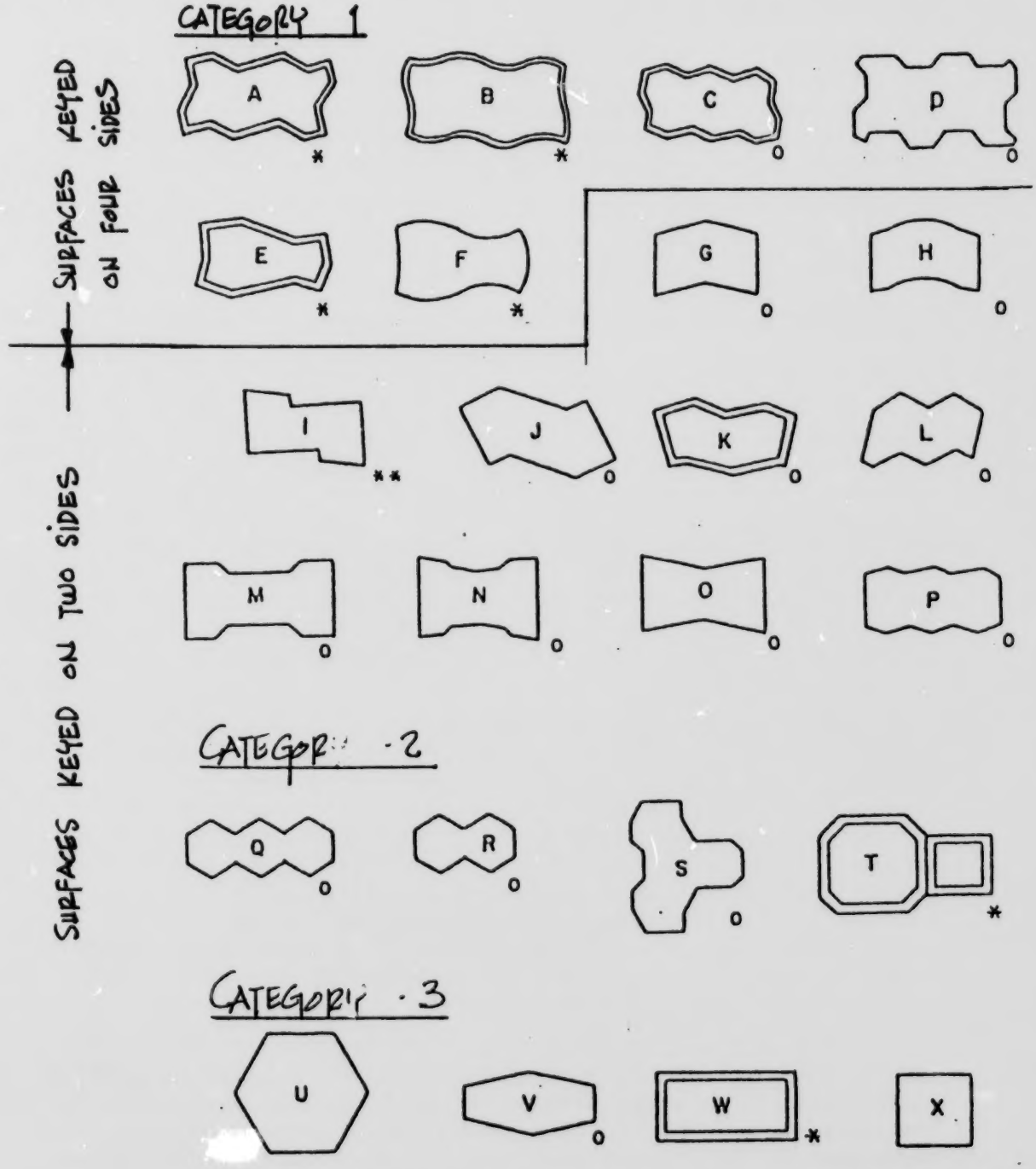


FIGURE 2.2

GEOMETRICAL SHAPES OF CONCRETE PAVING BLOCKS



o : LAID ONLY IN RUNNING BOND
* : LAID IN NUMEROUS PATTERNS
** : LAID ONLY BASKET WEAVE

(SHACKEL-1980)

The advantages and disavantages of each type of block must be evaluated with respect to the ease of fabrication, the variety of patterns that can be laid using the particular shape and the resulting behaviour of the pavement surface when subjected to traffic.

During fabrication, the dimensions of a block are easily controlled when the longitudinal cross-section, and especially the transverse section, remain unchanged. It is more difficult to respect dimensional tolerances when the block has curved surfaces and the mould wears with use.

Three pavement patterns, illustrated in figure 2.3, are in current use. These are:

- The stretcher or running bond,
- The herringbone pattern or basket weave, and
- The marquetry or parquetry pattern.

It is important that the shape of the block be readily adaptable to the selected pavement pattern. With the stretcher bond arrangement of figure 2.3, it is possible to position the neighbouring blocks precisely with respect to each other - as much in the longitudinal direction as transversely. But it is a little more difficult to lay blocks precisely in a herringbone pattern since the overall direction of this pattern can affect joint dimensions as work progresses. And finally, definite skill is required for the parquetry pattern since several directions are developed that can affect the joint dimensions.

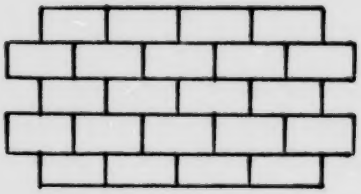
During laying, certain models of paving blocks such as those of type G, H and K must be turned 180° to create the desired pattern while those of type I and J must be inverted for each alternating row. The choice of paving block and laying pattern must be suitable to the competence of the workmen. Special attention must be given to the layout pattern: a stretcher bond pattern poses difficulties in curves, because it is necessary to strayaway from the pattern to reorient the rows; on the other hand, the herringbone pattern makes it possible to follow the geometry of the road without the need for reorientation at curves or intersections.

The behaviour of the pavement in service can be influenced by the form of the paving block and by the laying pattern used. With a variable transverse section, the paving blocks are more susceptible to be ruptured under load; and with reference to laying patterns used, the stretcher bond pattern is more susceptible to joint opening and consequently to the

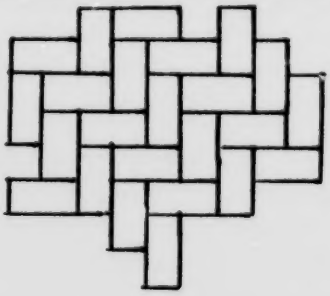
FIGURE 2.3

BLOCK PAVEMENT PATTERNS

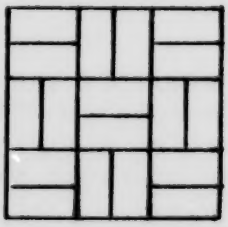
1. STRETCHER (RUNNING BOND)



2. HERRINGBONE (BASKET WEAVE)



3. MARQUETRY (BASKET WEAVE)



destruction of the structural integrity of the pavement. Finally, paving block displacement in the direction of the traffic can lead to important displacements of the whole pavement. Shackel's studies[1,3], supported by numerous observations, (1979-80), demonstrated that the herringbone pattern was the least susceptible to this kind of rupture.

2.1.1.2 The Bedding Sand

The sand laid between the paving blocks and the foundation serves to stabilize the paving blocks rather than redistribute stresses to the foundation. The ideal sand is clean and generally of medium size and angular. The presence of salt or clay in the sand makes laying difficult and hinders uniformity. A typical sand grading is shown in figure 2.4. The reduction in volume of the bedding sand at the time of compaction depends on the type of sand and its moisture content, as shown in figure 2.5[4]. Recent studies demonstrated that the properties of the bedding sand have a great influence on the behaviour of the block pavement when subjected to traffic.

2.1.2 Foundation Layers

The supporting foundation layers constitute the body of the pavement and are the structural elements responsible for transmitting loads to the subgrade. When the foundation consists of granular material, it should be well graded to provide high density and stability after compaction. The exclusive use of granular material is generally insufficient for heavy traffic loads as in the case of pavement for container terminals. Here, it is generally advisable to use cemented materials for the base layer and granular material for the foundation layer to provide uniform support and a well controlled profile.

Two types of cementitious material are generally used: Asphalt cements and Portland cements. Because of the heavy surface loads, asphalts are rarely used for industrial pavements; while Portland cements are currently used, notably in lean concrete bases, cement stabilized granular materials, and soil cement.

In Portland cement stabilized bases, the cement content increases as it passes from lean concrete to soil cement. The strengths of these mixtures diminish as the filler materials pass from well-graded concrete aggregate to a natural fine-grain soil (silt and clay). There are many slight differences between the three types of cement stabilization material and it is necessary to be precise when specifying

FIGURE 'A

GRADING OF BEDDING SAND RECOMMENDED BY HODGKINSON (AUSTRALIA)

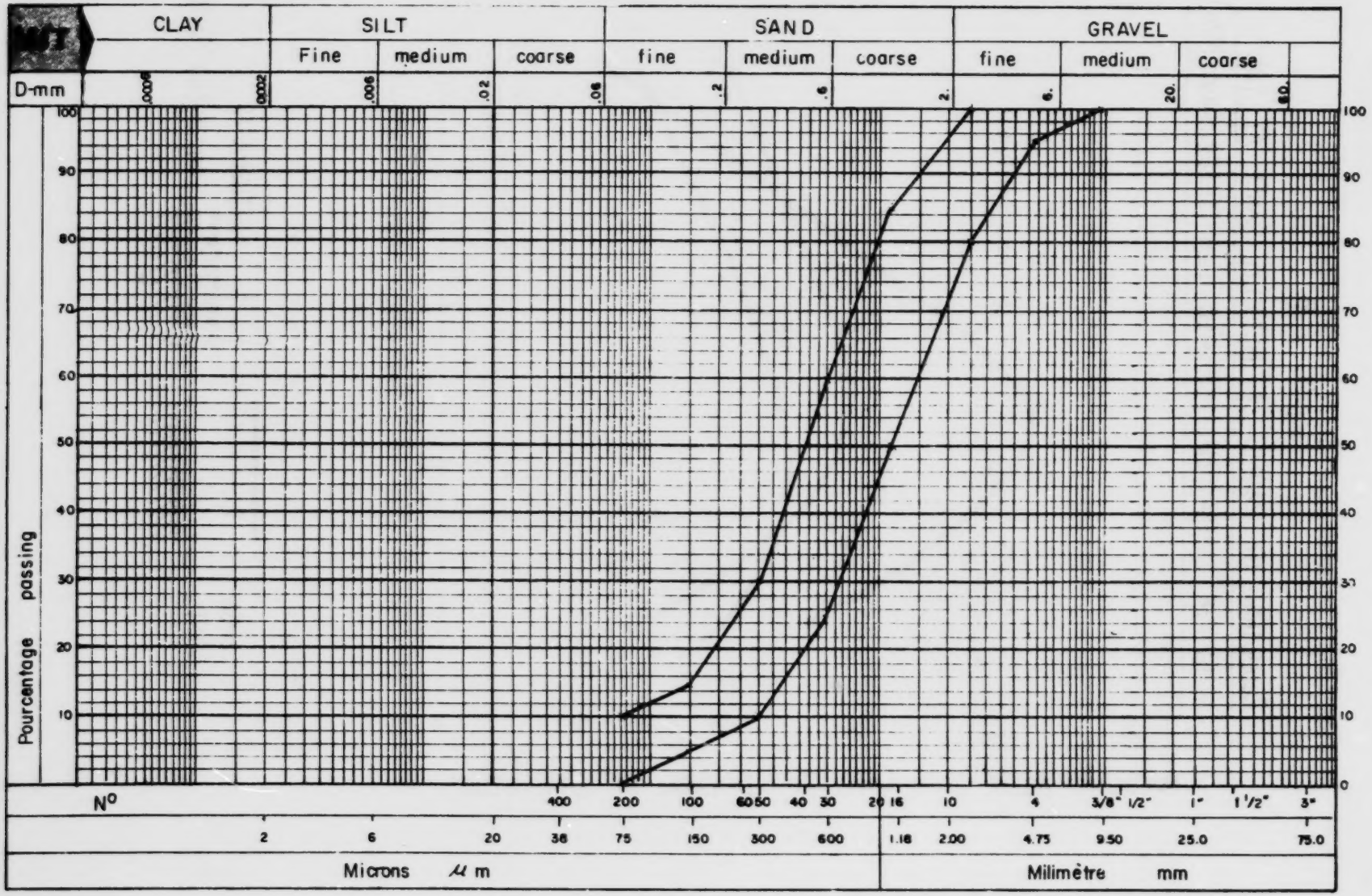
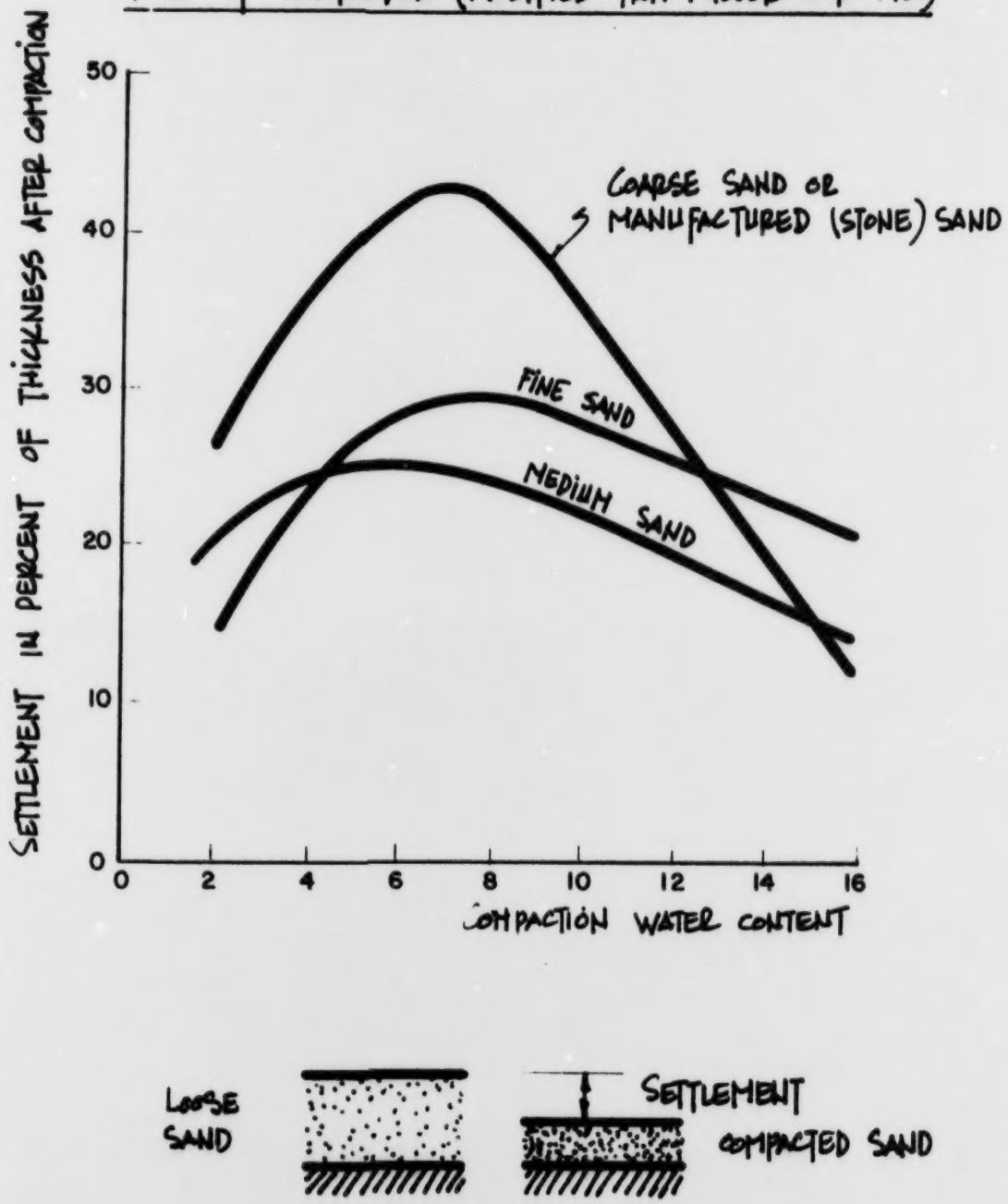


FIGURE 2.5

REDUCTION IN VOLUME OF BEDDING SAND AT THE TIME OF COMPACTION (MODIFIED FROM MILLER-COOK 1980)



such materials. The selection must be made by optimising the cost of aggregate as a function of the cost of cement necessary to achieve a given strength. In certain cases, the cost of concrete aggregate can be so high as to make the use of lean concrete prohibitive. In those circumstances it might be more economical to specify a less expensive aggregate which requires a higher cement content but less control. The curing of Portland cement bases are generally not affected by the laying of paving blocks and might even be helped since these blocks induce only slight stress in the stabilized layer and slow down the drying of that layer.

2.1.3 Sub-Grade

Pavement sub-grades are generally controlled draining materials with characteristics providing adequate strength to the road structure. However, in port areas the sub-grade is often constructed of hydraulic fill materials susceptible to settlement with time. It might therefore be necessary to accelerate these settlements by consolidation using additional fill or by dynamic compaction. One might also consider a temporary pavement to be followed by a permanent installation after traffic has created initial short-term settlements.

2.2 Manufacture and Placement of Concrete Paving Blocks

Although the machines that produce hollow concrete blocks can be adapted to the production of paving units, special equipment has been developed to provide greater control in their dimensioning. Variations can now be limited to ± 0,5 mm and these are more than adequate to meet usual specified tolerances of ± 3 mm.

It is generally not necessary to have skilled labour lay paving block. The procedure is relatively simple:

a) Spread out and level a layer of damp bedding sand without compacting it;

b) Lay the paving blocks to the desired pattern;

c) Compact with a minimum of 2 passes using a vibrating plate compactor applied to the surface of the paving blocks;

d) Sweep a fine sand into the joints;

e) Compact again with a minimum of two passes to densify the sand in the joints and thus provide firmness to the block pavement;

f) If necessary, repeat steps d) and e).

The "Cement and Concrete Association of Australia"[5] (1978) suggests an appropriate vibrating plate compactor must have a surface varying from .35 to .50 m^2 to cover about 12 paving blocks and capable of applying a force on the order of 16 to 20 mN at a frequency of 75 to 100 Hz. Experience has shown that vibrating compactors, steel-wheel or pneumatic compactors (vibrating or not) can also be used and that, in certain cases, they are even preferable.

In order to insure an adequate support of the paving blocks along pavement edges, sidewalk curbs or structural elements developed for this purpose can be used. Since lateral confinement must be ensured, the base and foundation layers should be spread under the entire width of the lateral confining elements.

As with other types of pavements, block pavements must be equipped with an adequate system of surface and sub-grade drainage. Given that the sand joints self-seal with fine particles after a certain time, it is generally sufficient to ensure surface drainage by adequate sloping.

2.3 Theory of paving stone road behaviour

Barber and Knapton[6] undertook a test program to better define paving block behaviour under load. The tests were done in the laboratory under controlled conditions on a section of roadway constructed of rectangular paving units laid in herringbone pattern. Based on their results, they described the behaviour of block pavements:

a) When such a road undergoes repeated loads, the relative movement of neighbouring blocks contribute to compaction of the joint sand. From the outset the rigidity of the layer increases rapidly under the effect of traffic.

b) Providing no rupture takes place, the interlocking capacity of the pattern develops with the deformation of the road; and as the interlocking capacity is raised, the road becomes more rigid and better able to distribute loads.

c) The pavement deformation Y is a function of the number of applied loads (N), in the form: $Y = AN^n$ where n is an exponant less than 1 and (A) is the proportionality constant which depends on the amplitude of deformations during the first load cycles and which diminishes generally with an increase in the bedding sand thickness.

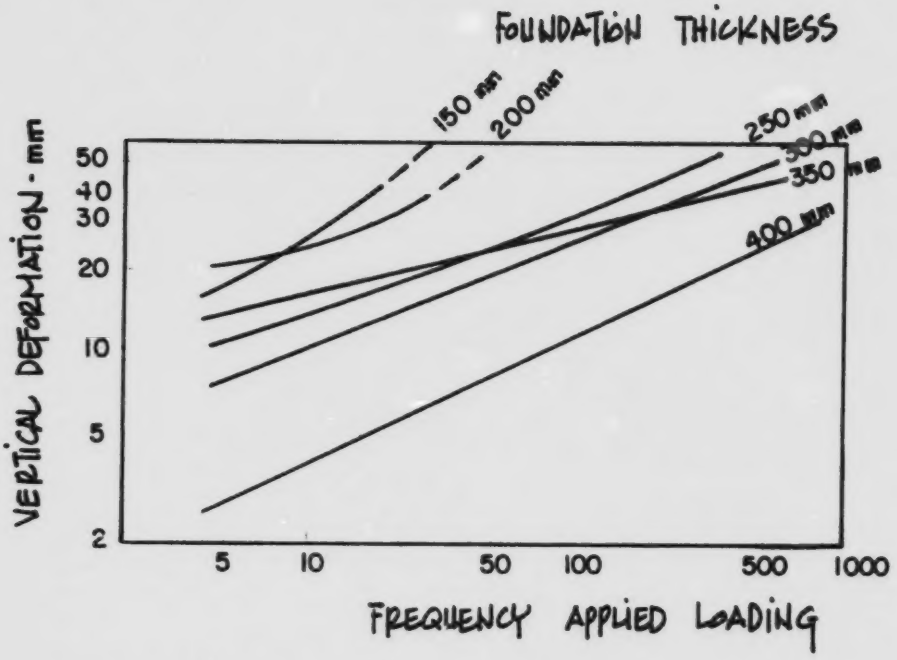
d) When the layer of bedding sand is not too thick (50 mm), a load of 6,35 tons will create deformations and shearing forces at the joints until rupture by punching of

the pavement, the bedding layer and the foundation. Where the shearing strength at the joints is greater than the inherent strength of the paving unit, the latter will split in two.

e) When the bedding layer is thick (150 mm and more), the shearing constraints in the joints will be weaker and important deformations will take place. These deformations will increase the interlocking capacity of the paving blocks and produce an annular swelling around the loaded zone. As the bedding sand becomes thicker, the annular swelling decreases. On the other hand, the swelling produces an arch effect which increases the pavement rigidity and per units distribution of the loads over a greater surface. Scaling of block corners close to areas of applied loads indicate the presence of high stresses. Finally, as illustrated in figure 2.6,there seems to exist a linear relationship between the observed settlement on the right of the loaded zone and the number of loads, when expressed logarithmically.

FIGURE 2.6

RELATION BETWEEN FREQUENCY OF APPLIED LOADS AND VERTICAL DEFORMATION



NOTE: TEST WITH RUBBERIZED (25 mm) STEEL PLATE (ϕ = 200 mm) AND 6.35 TONS LOAD.

3. DESIGN METHODS

There are no completely satisfactory methods for designing block pavements. While experience has led, in certain cases, to empirical dimensions that have proved adequate, some semi-analytical methods have been based on flexible pavement calculations.

3.1 Empirical Methods

Twenty to thirty years ago, the dimensioning of block pavements was a matter of referring to a catalogue for the appropriate structural type. These road sections were based on European experience and gave the required thicknesses for bedding, foundation and sub-grade without any indication of standards that related to the quality of materials and compaction. Shackel [7] suggests the reason for entrusting design to such a vague method was the small variations found throughout the Continent in strength and dynamic modulus of foundation materials; and with a water table often at the surface, the support and foundation layers stayed saturated. Such uniformity helped in the use of this kind of handbook. Table 3.1 lists the pavements in the German standard and figure 3.1 illustrates the Dutch pavements. Comparison of the two standards shows greater latitude in choice of materials by the Germans. Caution should be exercised in attempting to integrate design methods for block pavements used by other countries where climate and traditions may be different.

3.2 Semi-empirical Methods

The expression "semi-empirical" will be used here to mean design methods resulting from both theory and experimental tests. There are two distinct approaches: the first attempts to define the equivalent factor between the rigidity of the wearing surface constructed of paving blocks and bedding sand and known road paving materials and then to dimension the road with the help of traditional methods. The second does not take into account the structural value of the paving blocks but uses charts that permit dimensioning the structural layers as a function of the CBR load handling index of the soil.

3.2.1 Equivalent Thickness Methods

The equivalent thickness method for design is used in England and Australia. In England, the equivalent factor is determined with the help of static load tests on block pavement and on the materials under consideration; while in Australia, equivalent factors are based on the depth and the radius of

TABLE 3.1

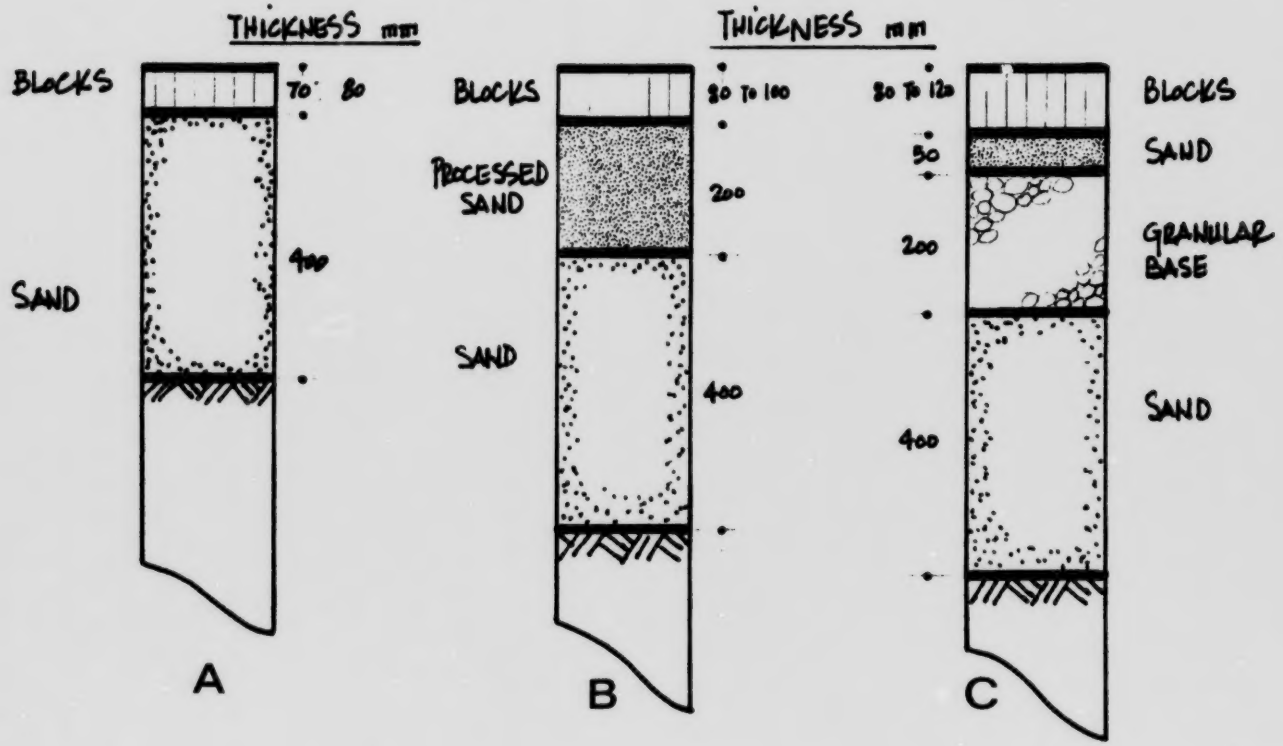
GERMAN STANDARD FOR BLOCK PAVEMENTS

NATURE OF TRAFFIC	PAVEMENT * mm	FOUNDATION ALTERNATIVE SOLUTIONS
Residential Streets and Parking Lots	60 - 80	a) 150 mm - Gravel Macadam b) 200 mm - Crushed Stone c) 250 mm - Gravel
Collector Roads Industrial Roads Autobus Stops Truck Stops Warehouse Loading Docks	80 - 100	a) 150 mm - Lean Concrete b) 100-150 mm - Bituminous " c) 250-300 mm - Gravel d) 180-220 mm - Crushed Stone e) 150-180 mm - Macadam
Arteries Industrial Parks Autobus Parking Areas	100 - 120	a) 120-180 mm - Lean Concrete b) 120-180 mm - Bituminous " c) 200 mm - Crushed Stone d) 200 mm - Macadam
Heavy Industrial Traffic	100	a) 300-600 mm - Lean Concrete or Gravel b) 300-600 mm - Crushed Stone
Industrial Land	80	a) 300-500 mm - Gravel

* On bedding sand, 30-50 mm thick

** Pavements must be protected from frost by a sub-foundation layer

FIGURE ..1 DU..H STANDARD FOR BLOCK PAVEMENTS



- PEDESTRIAN PATHWAYS
- VEHICLE PARKING
- LIGHT TRAFFIC

- RESIDENTIAL STREETS
- RURAL ROADS
- ROADS WITH OCCASIONAL HEAVY VEHICLE TRAFFIC

- HIGHWAYS
- AUTOBUS STOPS
- INDUSTRIAL ROADS
- HEAVY VEHICLE ROADS

curvature of rutting as well as the vertical constraint transmitted to the soil as determined by accelerated laboratory load tests. In both cases, the equivalent factor is defined as the relationship of thicknesses (between the layer of material under consideration and the layer of reference material) which produce the same effects on the road.

In table 3.2a can be found the equivalent factors of materials currently used in England and Australia. In table 3.2b are equivalent factors specific to each criteria (rutting and constraints on the soil); and for comparison, table 3.2c gives the equivalent factors suggested by the Asphalt Institute for road pavements and those of Transport Canada for runways.

3.2.2 Statistical Method

The statistical method suggested by Schakel [1, 3, 7] is based on laboratory tests conducted at the University of New South Wales in Australia. These experimental road tests made it possible to reproduce the effects of a vehicule moving at 14 m/min. Tire movement was simulated by a series of seven hydraulic jacks with 20 cm plates making it possible to apply consecutive contact pressures of 600 kPa or 900 kPa. These last figures represent a 24 kN axle-load generally permitted on roads and that of container manipulation equipment. The different test series were undertaken to determine the influence of the following factors:

a) Thickness of the support layer,

b) Thickness of paving blocks,

c) Contact pressure,

d) Total load.

The test results permitted Schakel to establish, by multiple regression, relationships which exist between the minimum thickness required for the support layer and the following factors: load handling index. depth and radius of curvature of depressions, constraint transmitted to the foundation soil, and block thickness. The resulting equations were logarithmic polynomials that led to three charts making it possible to calculate the minimum thickness of the support layer necessary for different paving block thicknesses with respect to the criteria of radius of curvature of rutting and permissible constraints on the soil (figures 3.2 to 3.4). When the CBR load bearing index of foundation materials is less than 50%, it is necessary to modify the foundation thickness by some correction factor. Shackel proposed to use the chart of figure Notice that no correction is given when the

TABLE 3.2a

EQUIVALENCE FACTORS CURRENTLY USED IN THE DESIGN OF BLOCK PAVEMENTS

COUNTRY	MATERIAL	EQUIVALENT THICKNESS	EQUIVALENCE FACTOR
UNITED KINGDOM	Blocks (80 mm) on 50 mm sand bed	80 mm	1
	"Rolled asphalt"	160 mm	2
	Lean concrete	225 mm	2.8
	Dense Macadam	170 mm	2.1
AUSTRALIA	Blocks (100 mm)	100 mm	1
	Bituminous concrete	130 mm	1.3
	Crushed stone	250 mm	2.5

TABLE 3.2b

RELATED EQUIVALENCE FACTORS FOR VARIOUS CRITERIA

MATERIAL	EQUIVALENCE FACTOR		
	Related to depth of rutting	Related to radius of ruts	Related to soil constraints
Blocks 100 mm thick	1.0	1.0	1.0
Crushed Dolerite	2.0	2.8	2.1
Bituminous Concrete	1.5	1.1	1.4

TABLE 3.2c

EQUIVALENCE FACTORS CURRENTLY USED FOR DESIGN OF FLEXIBLE PAVEMENTS

AGENCY	MATERIAL	EQUIVALENCE FACTOR
TRANSPORT CANADA	- Crushed stone	1.0
	- Cement-bound macadam	1.5
	- Bituminous concrete (high quality)	2.0
	- Asphalt pavement	1.5
ASPHALT INSTITUTE	- Bituminous concrete	1.0
	- Granular material (high quality)	2.0
	- Granular (poor quality)	2,7

CBR index of the soil is greater than 50%, since above this value the required foundation layer thickness is little influenced by an increase in CBR index.

Thus, in order to use Shackel's charts it is necessary to choose appropriate criteria for the road behaviour. But it can sometimes be difficult to choose appropriate criteria, so Shackel proposes the series of criteria represented in table 3.3. According to the author, these are conservative and permit a road design appropriate to normal use under three conditions of traffic. Applying these to the previously described method, it is possible to derive charts which make it possible to dimension the layer of support from the CBR index of the sub-grade layer. These charts are presented in figures 3.6, 3.7 and 3.8.Note that this method assumes implicity that the road is made up of a crushed stone foundation layer of good quality a 30 mm-thick layer of bedding sand after compaction and paving block. This method does not take into account the number of applications of axial loads as in the case of flexible roads because, according to Shackel, paving block have a tendancy to interlock, to stiffen and to stabilize after a short period of use.

3.2.3 PCA Method Adapted for Block Pavements with Soil-Cement Base

The method of dimensioning soil-cement roads proposed by Portland Cement Association [8] was adapted by the "British Cement and Concrete Association" for block pavement design. The adaptation used an equivalent factor of 2,25 between soil-cement and paving block, regardless of the thickness. This method is valid for axial loads varying between 9 and 45 tons and for CBR load handling indices from 2 to 25%.

The PCA method was also adapted by Tait[9] in New Zealand but with about half the equivalent factor to 1.1. The soil-cement road method of dimensioning, requires the following information:

1. Resistance of the soil or combination of soil and foundation;

2. Data related to vehicle loads prior to traffic as well as on the road.

However, information related to the strength of the soil cement is not absolutely necessary.

USCE research showed that the radius of curvature of a soil-cement road under the greatest load, depends on the

FIGURE 3.2

CHART FOR DETERMINATION OF MINIMUN THICKNESS OF SUPPORT LAYER IN RELATION TO BLOCK THICKNESS AND DEPTH OF RUTTING

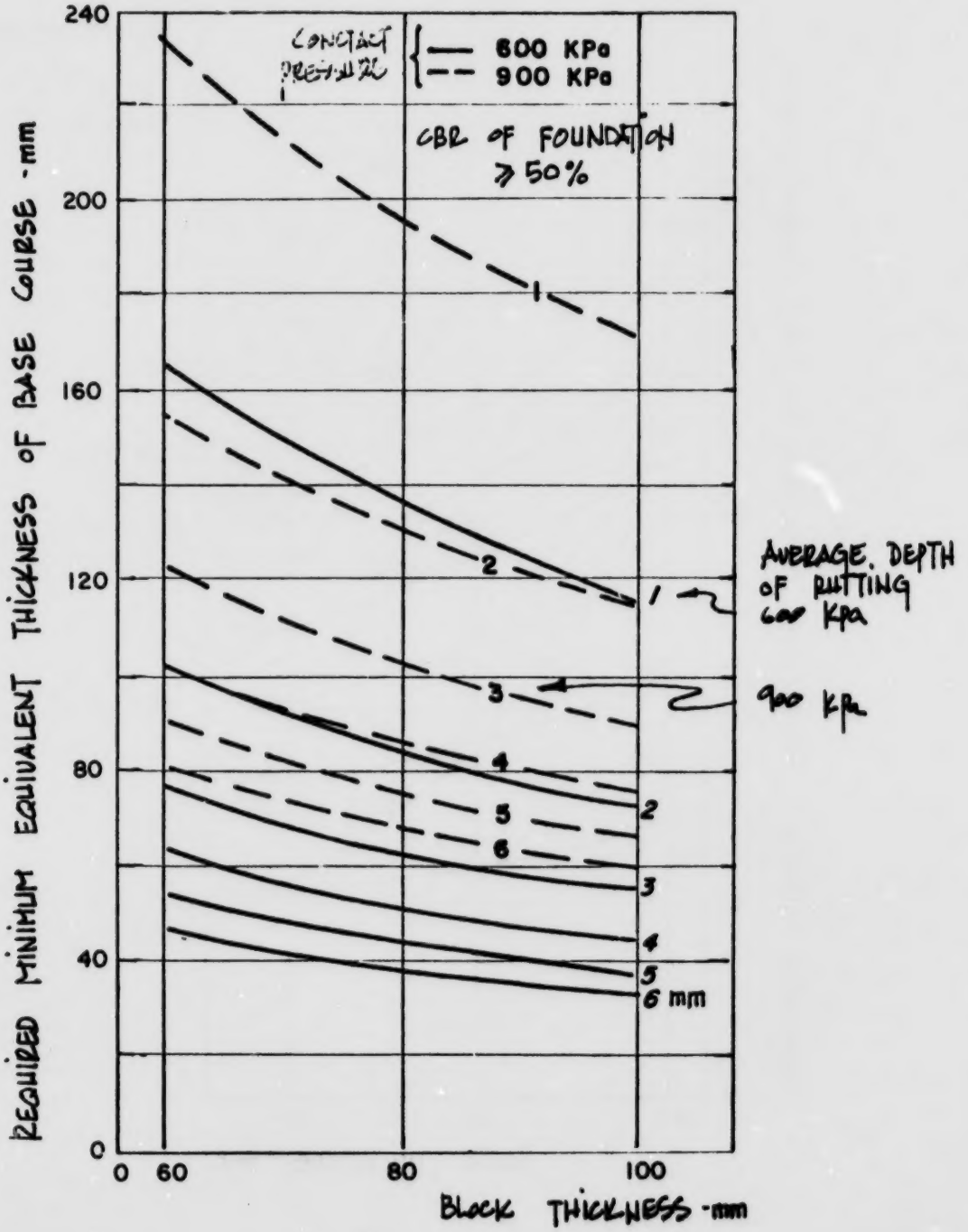


FIGURE 3.3

CHART FOR DETERMINATION OF MINIMUM THICKNESS OF SUPPORT LAYER IN RELATION TO BLOCK THICKNESS AND RADIUS OF CURVATURE

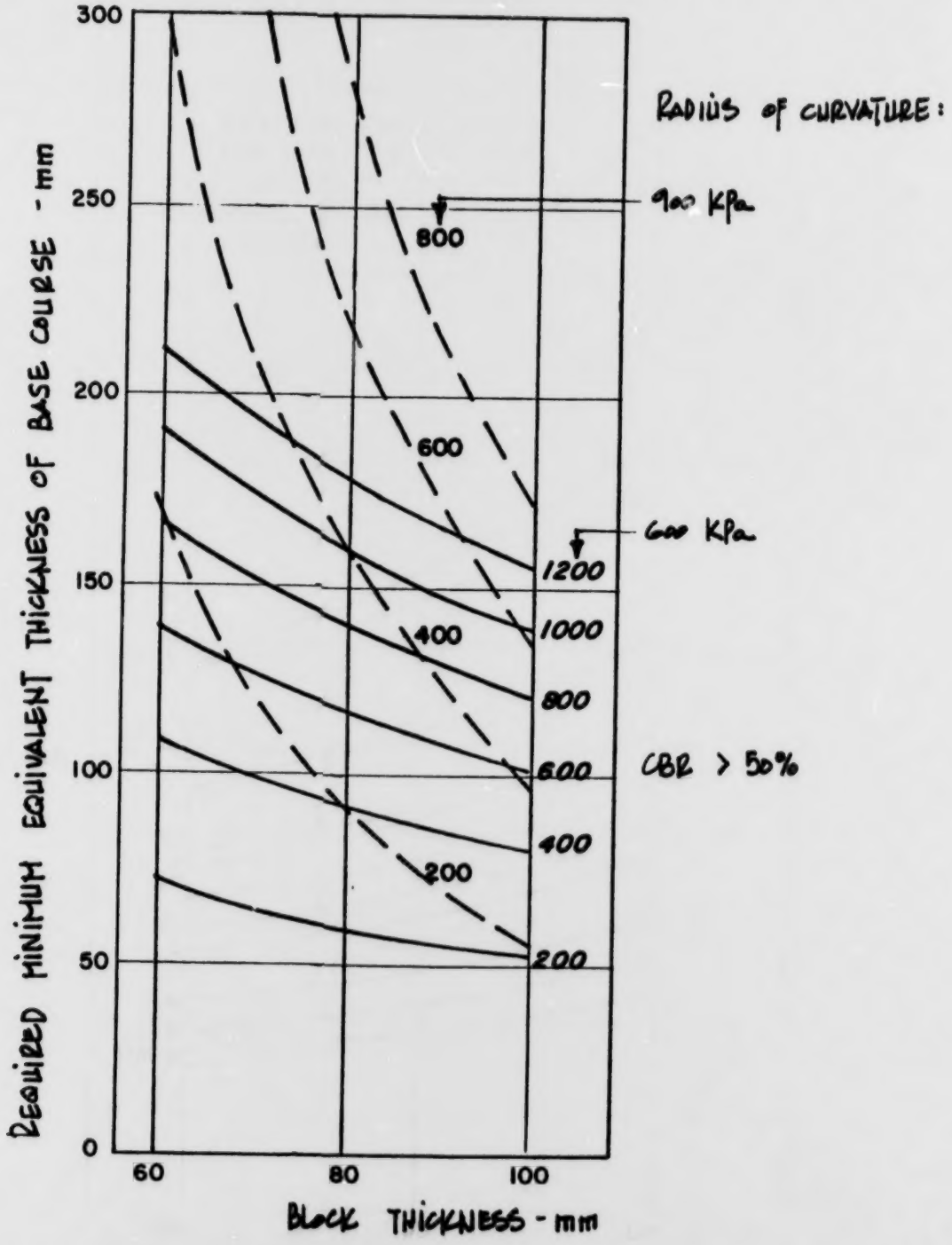


FIGURE 3.4

CHART FOR DETERMINATION OF MINIMUM THICKNESS OF SUPPORT LAYER IN RELATION TO BLOCK THICKNESS AND STRESS ON THE SUBGRADE

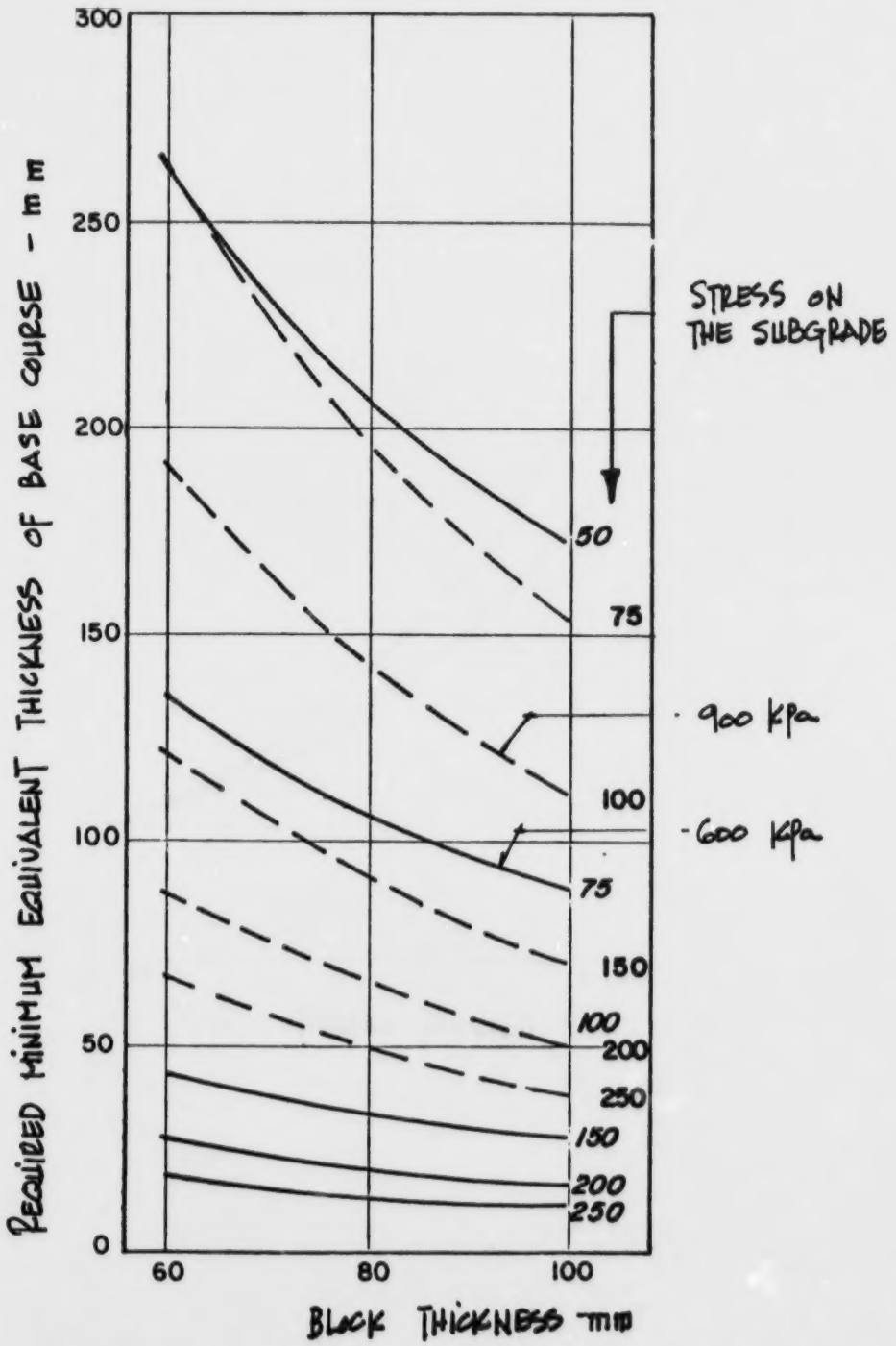


FIGURE 3.5

CORRECTION FACTOR RELATED TO SOIL BEARING CAPACITY (CBR)

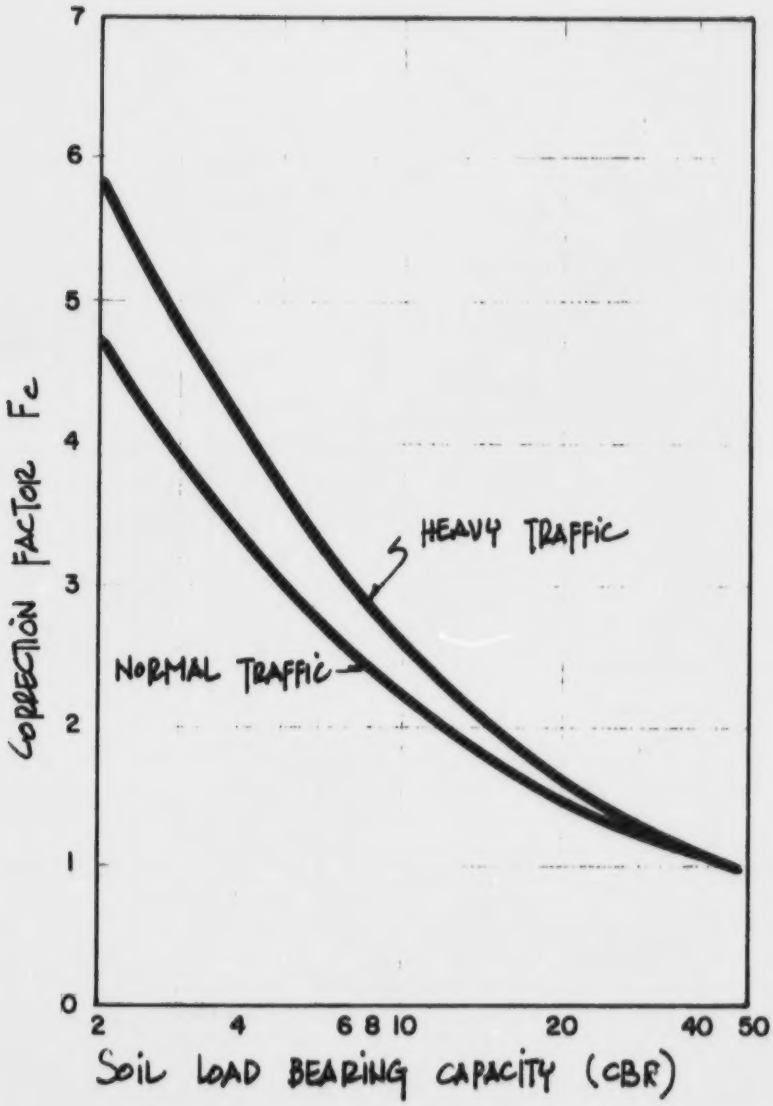


TABLE 3.3

BEHAVIOUR CRITERIA OF BLOCK PAVEMENTS
(Sheckel, 1980)

NATURE OF TRAFFIC	TYPE OF ROAD	BEHAVIOUR CRITERIA		
		DEPTH OF RUTTING	RATIO OF RADIUS % CURVATURE	PRESSURE
Light	Residential Street	6.0	100	600
Heavy	Artery	3.0	200	600
Heavy	Industrial	3.0	100	900

FIGURE 3.6

CHART FOR DETERMINATION OF FOUNDATION THICKNESS - LIGHT TRAFFIC

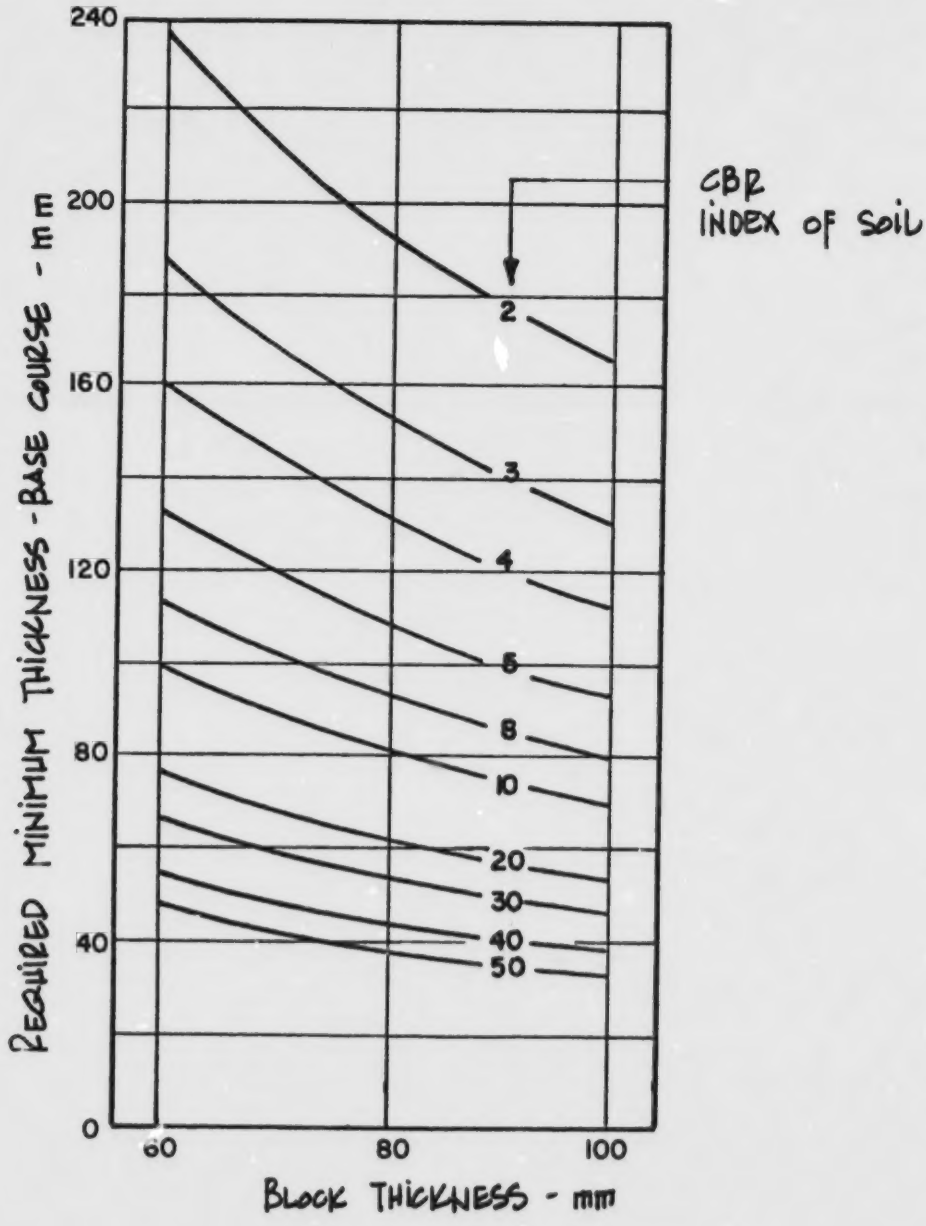


FIGURE 3.7

CHART FOR DETERMINATION OF FOUNDATION THICKNESS - HEAVY TRAFFIC

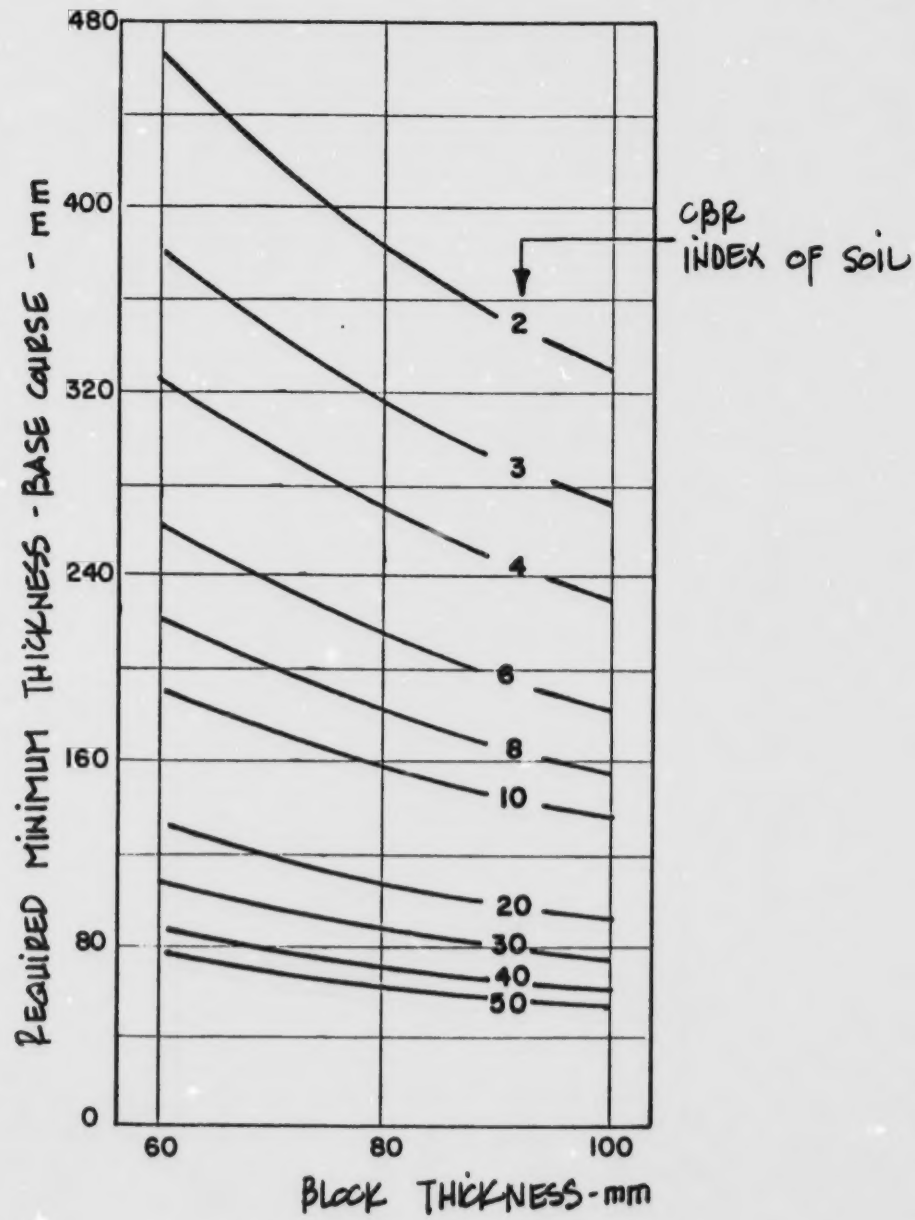
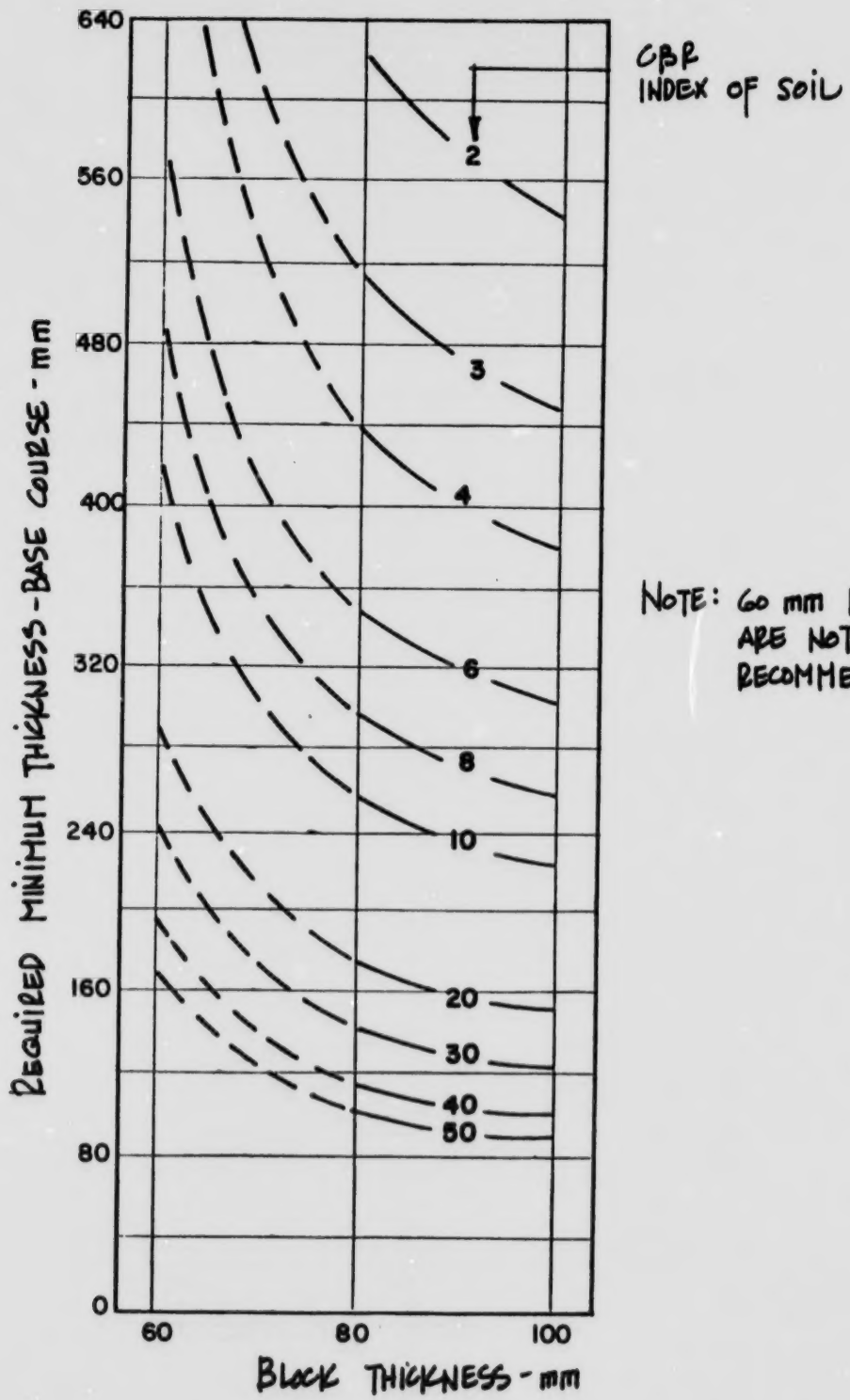


FIGURE 3.8

CHART FOR DETERMINATION OF FOUNDATION THICKNESS - HEAVY INDUSTRIAL TRAFFIC



greatest flexural deformation and is for all practical purposes constant whatever the type of crushed material used. It was also shown that using the radius of curvature for design was preferable to that of compressive strength or any other kind of strength related to the type of mix.

3.2.3.1 Determination of Bearing Capacity of Sub-Grade

Bearing capacity of soil is defined by the Westergaard reaction modulus and is determined by plate tests. The reaction modulus being the quotient of the unit load applied to a 30 cm diameter plate divided by the resulting deformation of the adjacent soil. It may also be defined as being the quotient of the applied load on the plate divided by the volume of displaced soil. When it is not possible to perform tests, one can, as suggested by the Portland Cement Association [10], use figure 3.9 to evaluate the reaction modulus. But if the soil-cement rests on a non-treated layer of granular material, one can evaluate the global reaction modulus of granular material and of soil using table 3.4.

3.2.3.2 Traffic Loads

The four following parameters govern the effects of traffic on the road:

1. The maximum wheel load,
2. The tire surface area in contact with the road,
3. The spacing of twin wheels,
4. The number of load passes.

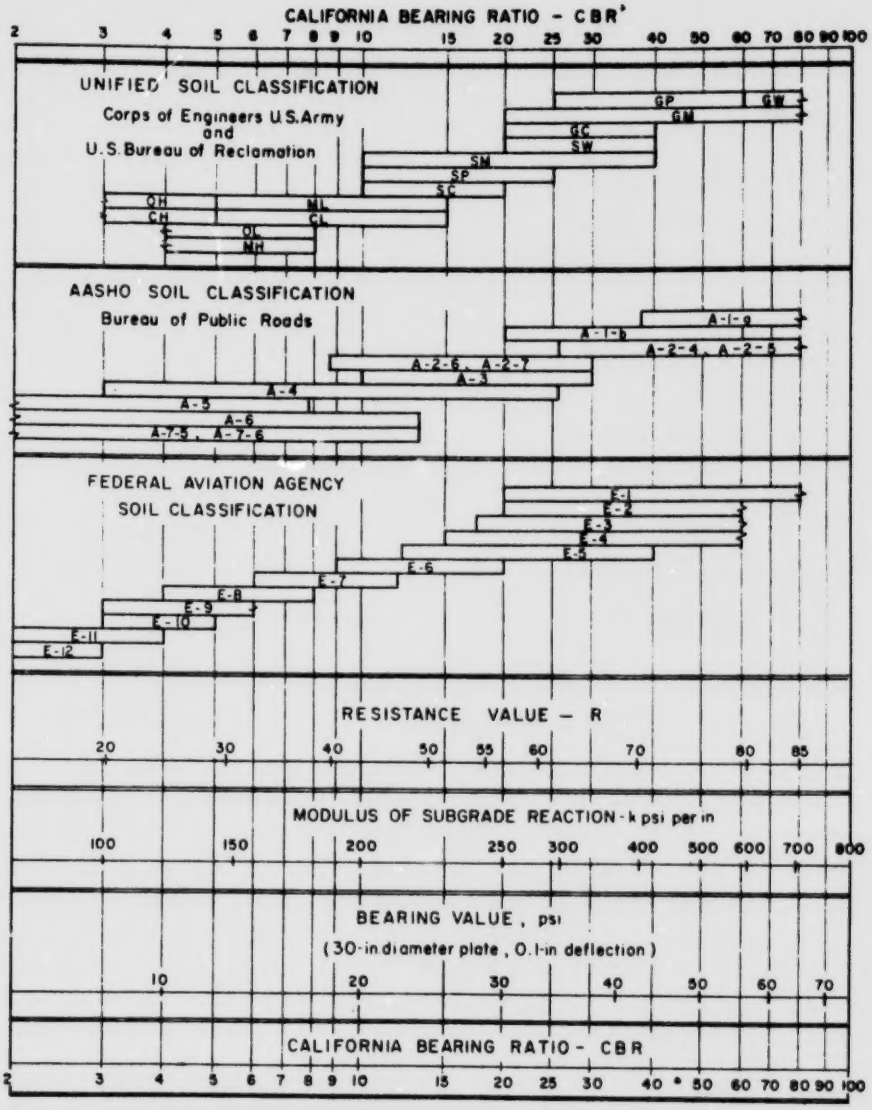
Road dimensions are usually a function of the heaviest vehicule. However if the predominant traffic is other types of vehicles almost as heavy, then the influence of these vehicles must be verified. The wheel-load is generally taken as half the maximum axle-load. The contact surface area of the tire is defined as the area of tire imprint and is independant of the tire groove pattern. If imprint measurement is impossible, it can be calculated by dividing the wheel load by the tire pressure.

The influence of twin wheels or in-tandem grouping can be found by determining the equivalent single wheel-load - by calculation or by using charts (figure 3.10).

3.2.3.3 Determination of Required Soil-Cement Layer Thickness

The required soil-cement layer thickness, without paving blocks, can be determined from the chart on figure 3.11 or by using the following formula:

FIGURE 3.9
APPROXIMATE INTERRELATIONSHIPS OF SOIL CLASSIFICATIONS AND BEARING VALUES



*Sources for values shown are given in PCA Soil Primer

TABLE 3.4a

EFFECT OF UNTREATED BASE COURSE THICKNESS ON MODULUS OF SUBGRADE REACTION OF SUPPORTING SOIL

SUPPORTING SOIL K VALUE		K MEASURED ON TOP OF BASE COURSE (UNTREATED)							
		10 CM/4 IN		15 CM/6 IN		22.5 CM/9 IN		30 CM/12 IN	
LB/IN^3	Kg/CM^3	LB/PO^3	Kg/CM^3	LB/PO^3	Kg/CM^3	LB/PO^3	Kg/CM^3	LB/PO^3	Kg/CM^3
50	1.38	65	1.79	75	2.07	85	2.35	110	3.04
100	2.77	130	3.59	140	3.87	160	4.42	190	5.26
200	5.54	220	6.09	230	6.36	270	7.47	320	8.85
300	8.30	320	8.85	330	9.13	370	10.24	430	11.90

TABLE 3.4b

EFFECT OF CEMENT STABILIZED BASE COURSE THICKNESS ON MODULUS OF SUBGRADE REACTION OF SUPPORTING SOIL

THICKNESS OF CEMENT STABILIZED LAYER		K MEASURED ON TOP OF THE STABILIZED LAYER	
IN	CM	LB/IN^3	Kg/CM^3
4	10.0	300	8.30
5	12.5	450	12.45
6	15.0	550	15.22
7	17.5	600	16.60

K: 100 LB/IN^3

FIGURE 3.10

EQUIVALENT WHEEL LOAD CALCULATION

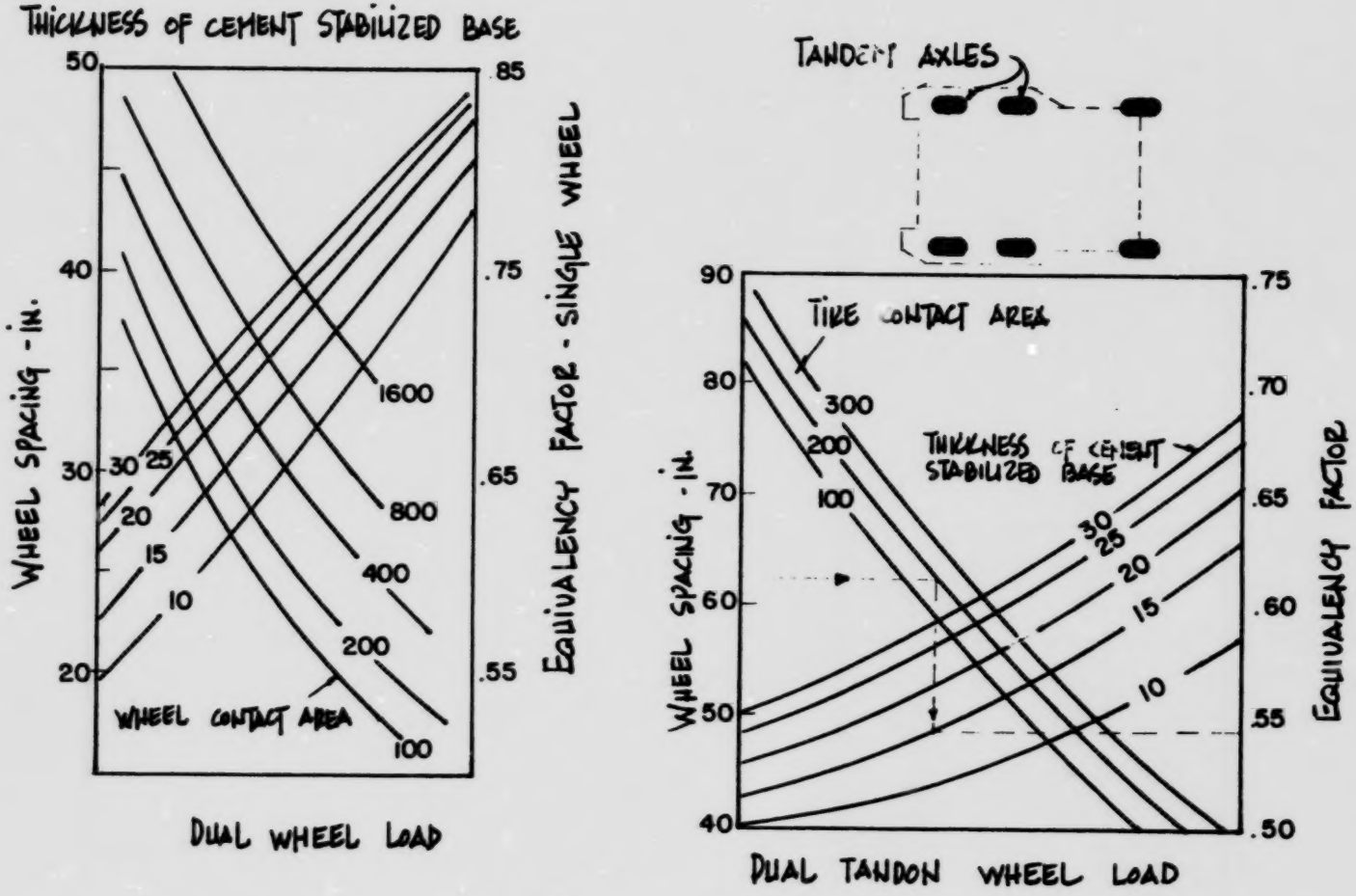
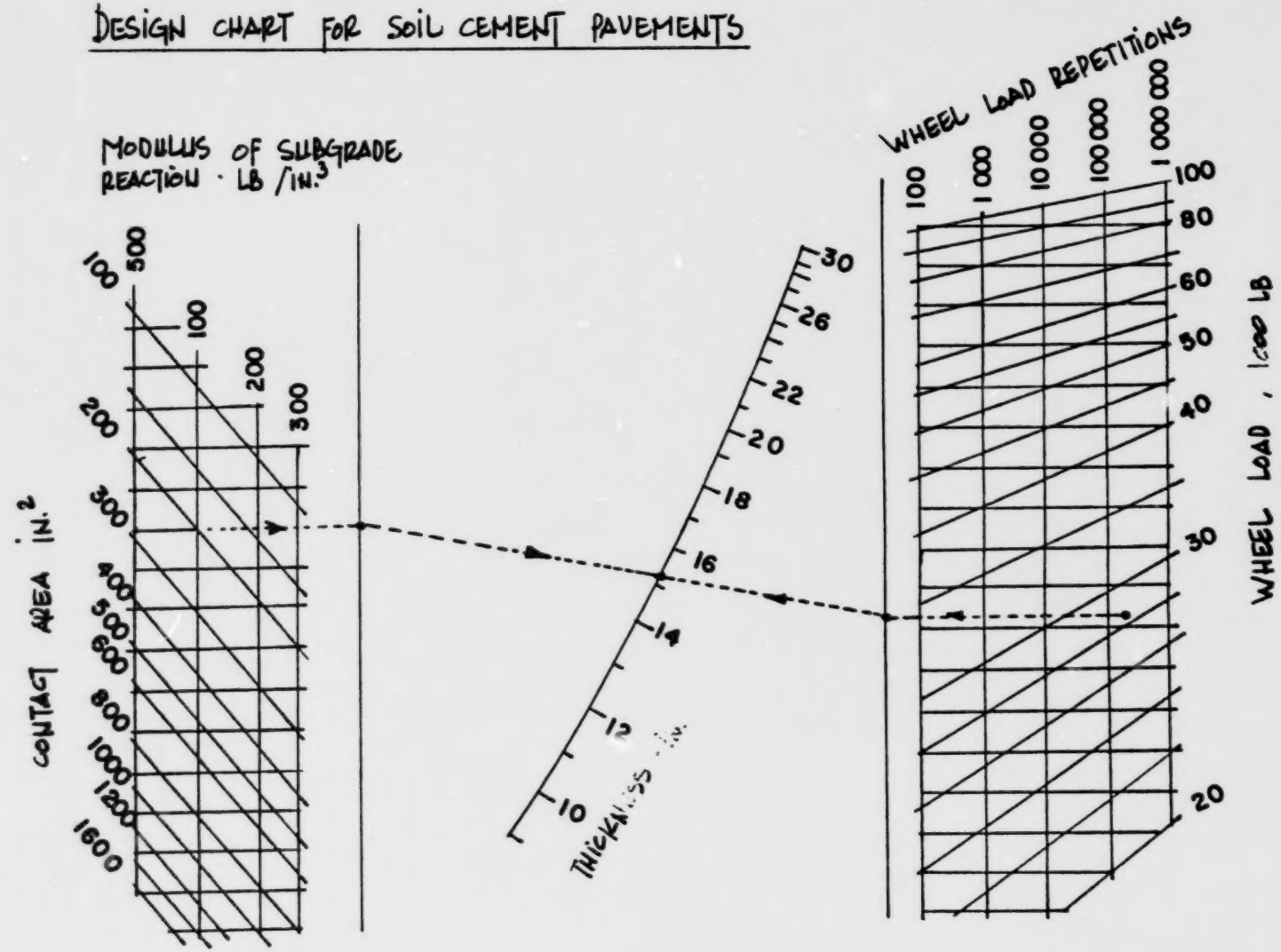


FIGURE 3.11

DESIGN CHART FOR SOIL CEMENT PAVEMENTS



$$fh = N^{1/40} * \frac{P}{a} * \frac{8.76277}{K^{0.3}}$$

avec $fh = (2.1h - 1)^2/h^{1/2}$

P : wheel-load
h : soil-cement layer thickness (in)
k : soil response modulus (lb-f/in^3)
N : number of load applications
a : tire contact surface area (in^2)

Note that the chart in figure 3.11 was established using a modulus of elasticity of 400,000 to 500,000 psi for the soil-cement. Where 100 mm-thick paving blocks are used, their structural capacity can be considered as having an equivalent soil-cement thickness which must be subtracted from the layer thickness found in the chart. This would be 225 mm or 110 mm, depending on whether the English or New Zealand Method was being used.

3.3 Semi-analytical Method

The method proposed by Barber and Knapton [11] is based on the road analysis developed by Ullidtz and Peattie [12] from a multilayered elastic numerical model. Several hundred pavement structures were analyzed, so that design charts could be established. Required road thickness was determined by consideration of two criteria for rupture:

1. The rupture or cracking due to material fatigue of the base layer.

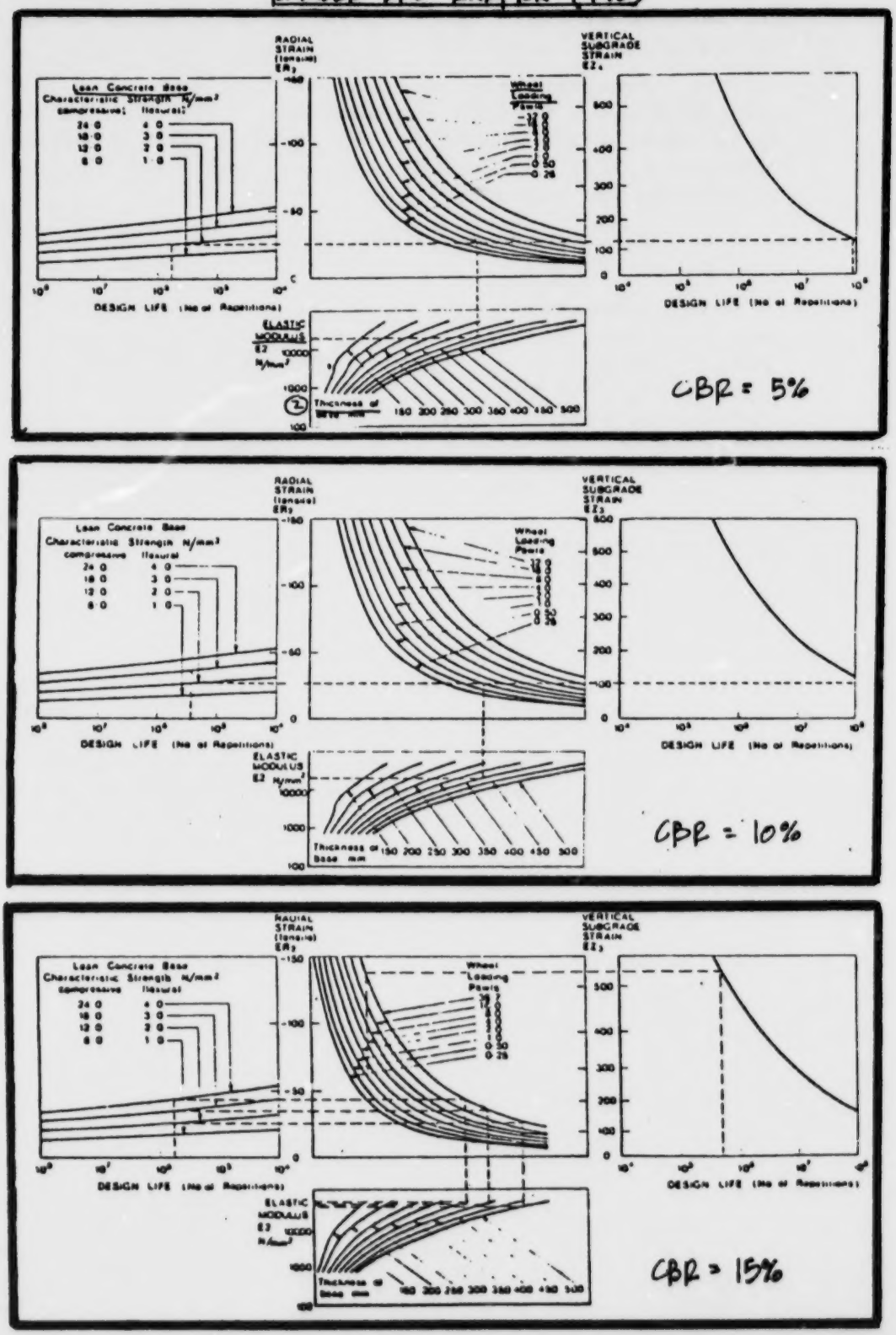
2. The progressive vertical deformation of the base soil.

The parameters used in order to quantify these criteria are the radial deformation in tension of the cement-stabilized foundation base layer and the vertical deformation in compression at the surface of the foundation soil.

Dimension charts are presented in figure 3.12 and are applicable only for roads having a block wearing course of 80 mm, a foundation layer stabilized by Portland cement and a sub-grade of granular material. Figure 3.12 shows three series of charts for specific CBR values of 5, 10 and 20%. To use the charts, it is necessary to provide the modulus of elasticity of the cement-stabilized material of the foundation, its compressive and tensile strengths, and to know the

FIGURE 3.12

DESIGN CHARTS FOR BLOCK PAVEMENT ACCORDING TO BARBER AND KNAPTON (1980)



traffic loads. Figure 3.9 contains the mechanical properties of some foundation materials. This table might be useful if laboratory-tests are not available. The pavement may be designed as follows:

1. Determine the pavement traffic loads and their average influence on road degradation. Note that in port handling these loads are expressed in PAWL (Port Area Wheel Load) where one PAWL, by definition, is a unit load of 12 tons having a damage factor of one.

2. Determining for each vehicle the number of anticipated passes (N_1, N_2,...N_m) during the anticipated road life.

3. Determining the number of unit loads (PAWL) to which the road will be submitted during its anticipated life (N PAWL). To do so, multiply the number of passes for each type of vehicle by its average degradation factor, then calculate the sum of degradations which would be caused by all vehicles (N PAWL = $\Sigma^n N_i$ X P average).

4. At the time of design, determine the pavement life span (D V), or the equivalent number of passes causing the anticipated degradation. That is divide N PAWL by the load causing the anticipated degradation at the time of design (Pd):

$$D\ V = N\ PAWL/Pd$$

5. Determine, using figure 3.12 charts the required thickness of the soil-cement layer to withstand the number of passes of anticipated load. The required thickness depends on the strength and elastic modulus of the soil cement and will limit either the vertical deformation at the soil surface or the radical deformation in the soil cement layer.

4. CONSTRUCTION OF TEST SECTIONS AND EVALUATION OF STRUCTURAL CAPACITIES

The experimental stage of this study comprises the construction of pavements and the structural and performance evaluations of five test sections subjected to identical load conditions by operating vehicles and container warehousing. The pavement sections differ in the materials used and thickness of layers. Here, the report will describe the characteristics of the test sections and the change in structural capacity with increasing pavement thickness, as follows:

a) Description of test sections,

b) Characteristics of materials used,

c) Method of evaluating structural capacity during construction,

d) Influence of different layers on the structural capacity.

4.1 Description of Test Sections

Five test sections were constructed at the Racine Terminal of the Port of Montreal. The configurations and dimensions of the sections are illustrated in figure 4.1.

The road structures and material composition are presented in figure 4.2. The sub-grade, identical and common to all the roads, is a dredging soil covered by about 1 m of mixed crushed stone waste and dredging soil dynamically compacted. The roads were dimensioned by one of two methods: for flexible pavements (USCE and rational method), or for soil-cement roads (PCA method). A brief description of each design follows:

Section A:

Starting from the platform, section A has a granular sub foundation layer of crushed stone, 0-75 mm (0-3"), 780 mm thick; a base layer of crushed stone, 0-20 mm (0-3/4"), 200 mm thick; a 30 mm layer of bedding sand; and a layer of paving block, 100 mm thick.

Section B:

This section is similar to section A in thickness of layers and type of materials except for the 220 mm thick layer of crushed stone and the thinner 80 mm paving block.

FIGURE 4.1

LAYOUT OF TEST SECTIONS

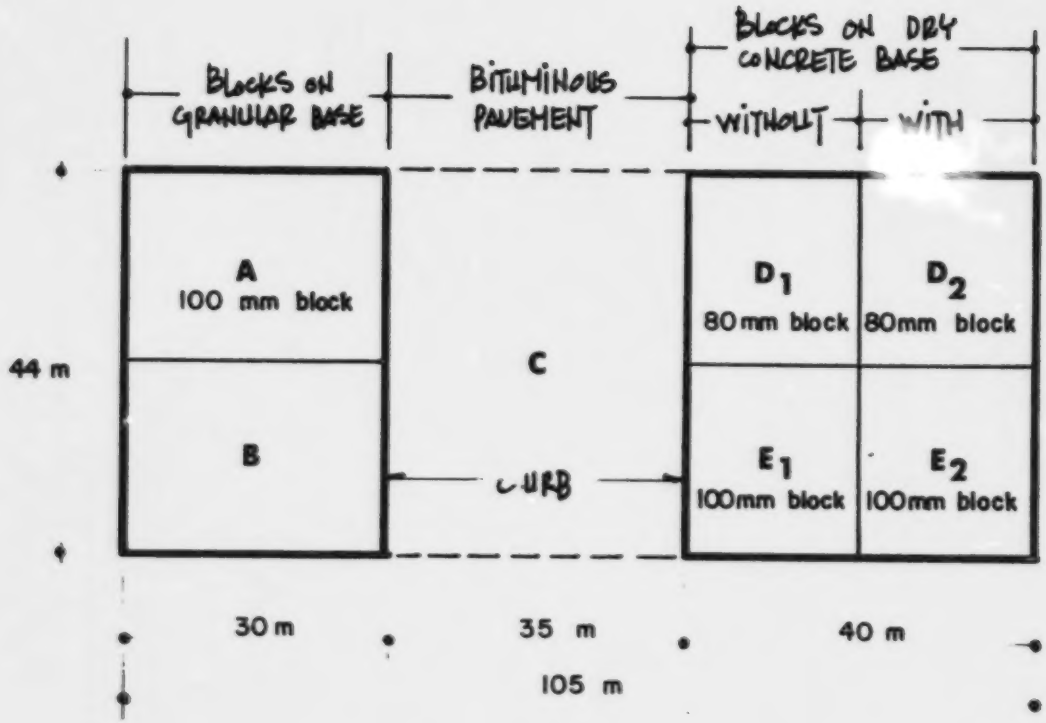


FIGURE 4.2

STRUCTURE AND COMPOSITION OF TEST PAVEMENTS

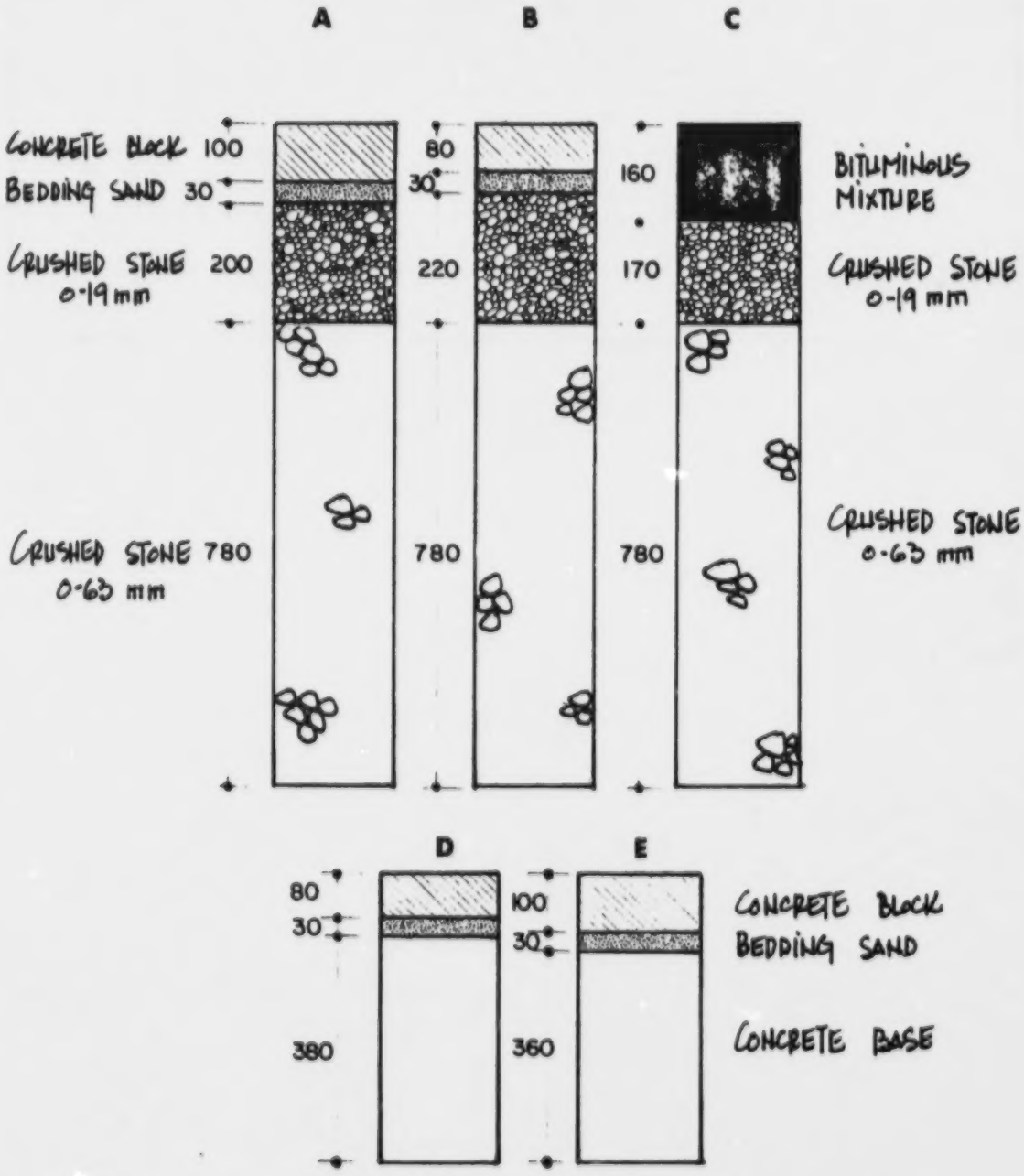


TABLE 4.1

TEST PAVEMENT MATERIALS

SECTION	PAVING BLOCK	BEDDING SAND	BITUMINOUS SURFACING	NO-SLUMP CONCRETE	CRUSHED STONE	
					0-20 mm	0-75 mm
A	X	X	-	-	X	X
B	X	X	-	-	X	X
C	-	-	X	-	X	X
D	X	X	-	X	-	-
E	X	X	-	X	-	-

Section C:

Here, a 160 mm-thick bituminous mix is used as the wearing course over a 170 mm-thick base layer of 0-20 mm (0-3/4") crushed stone which rests on a 780 mm-thick subfoundation of 0-75 mm (0-3") crushed stone.

Section D:

Section D consists of a 0-75 mm (0-3") crushed stone subfoundation, a no-slump concrete base, a layer of bedding sand and paving block. The layer thicknesses are 620 mm, 380 mm, 30 mm and 80 mm respectively.

SECTION E:

Pavement E is similar to Section D except for a change in thickness of 100 mm respectively for the no-slump concrete and paving block.

4.2 Characteristics of Construction Materials

4.2.1 Use of the materials in test sections: it can be seen that roads A and B are similar, as are D and E.

4.2.2 Material characteristics

The descriptive and mechanical characteristics of all materials used in construction were evaluated by standard tests and the results are presented here.

The grading test, the standard or modified Proctor density tests and the CBR load bearing test were used to describe and evaluate the granular materials; the composition or mix formula as well as compressive strength, density and slump were used to characterize the Portland cement concrete; and for bituminous concrete the Marshall test was used for mix proportions and control.

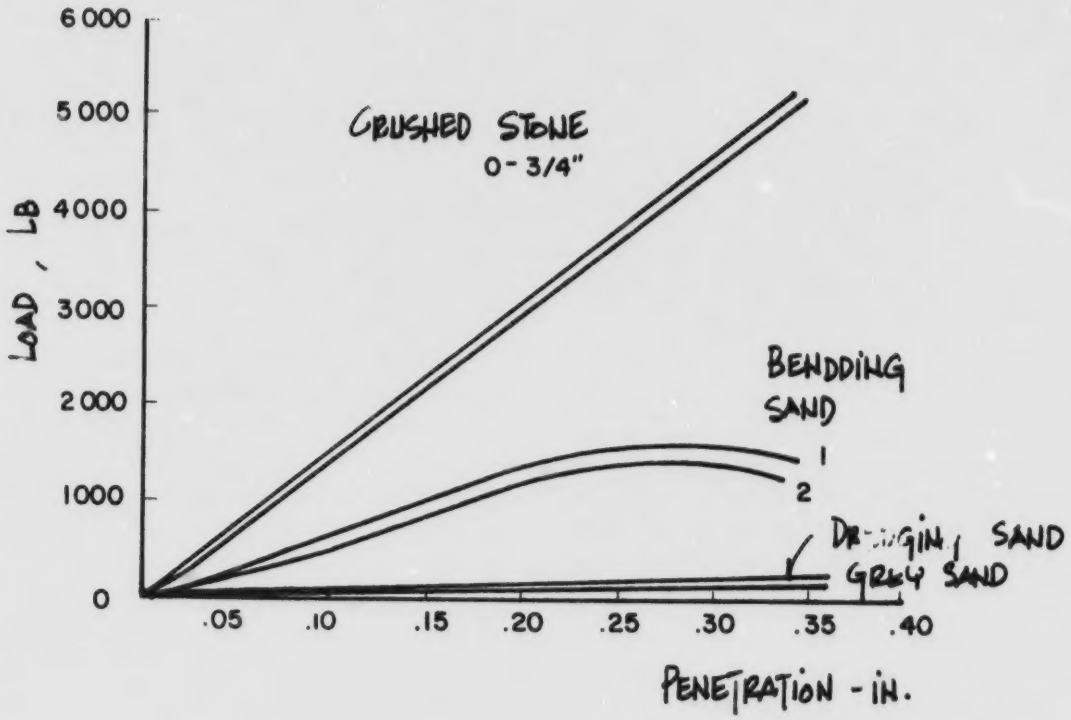
4.2.2.1 Dredging Sand Characteristics

a) Grading analysis

Sieve Size	Per cent passing		
	1	2	3
8	99	99	99
16	99	98	98
30	98	97	96
50	90	88	84
100	46	49	42
200	13,6	15,3	13,1

FIGURE 4.3

CBR TESTS OF MATERIALS- RACINE TERMINAL



b) Standard Proctor test (ASTM D-698, Method D)

Unit weight (lb/ft^3) 115,3

Optimum water content (%) 11,0

c) California Bearing Ratio (see figure 4.3)

Test load (lb) 15,0

Unit weight (lb/ft^3) 106,1, 107,2

Water content
. before test (%) 11,0
. after test (%) 15,9, 16,3

Swelling (%) 0

CBR load bearing capacity (%) ... 1,8, 2,0

Estimated Dynamic Modulus (psi).. 2700, 3000

4.2.2.2 Crushed stone characteristics (0-75 mm), Sub-foundation

a) Grading analysis

Test Dates

Sieve Size	4/8	18/9	18/9	24/9	30/9	30/9 *	30/9	30/9 *	7/10	9/10*	PORT'S SPEC.
3"	100	100	100	100	100	100	100	100	100	100	100
2"	99	98	100	95	100	98	98	98	100	100	82-100
1"	72	50	67	57	56	63	66	71	64	76	50-80
3/4"	53	40	59	47	46	57	57	58	52	65	-
4	28	13	27	17	22	27	21	23	23	35	25-50
16	14	5	13	9	13	15	9	10	8	22	11-30
50	9	4	8	6	8	9	6	5	4	16	4-18
200	7,8	3,0	5,9	4,1	6,4	6,5	4,7	2,0	1,8	13,0	2-8
	T	T	C	C	C	C	C	E	E	C	L L

Where,

T: model
C: control
E: evaluation
L: specification limits
*: after compaction

b) Modified Proctor Density (model)

Maximum density (lb/ft^3) 143,5
Optimum water content (%) 4,6
Specific gravity 2,63

4.2.2.3 Crushed stone characteristics (0-20 mm), Base

a) Grading analysis

Sieve Size	Test Dates 21/7	24/8	18/9	30/9	30/9*	13/10	15/10*	21/10	21/10*	22/10	22/10	SPEC.
1"	100	100	100	100	100	100	100	100	100	100	100	
3/4"	98	94	98	98	98	98	97	97	97	98	97	100
1/2"	88	74	74	80	79	81	77	76	78	72	75	68-93
4	59	45	41	45	45	45	46	42	43	33	45	35-60
8	40	31	24	28	31	30	33	27	30	21	29	-
16	27	21	15	18	21	20	24	16	19	14	19	19-38
50	15	12	9	10	12	11	13	8	9	7	11	9-17
200	9,8	7,8	6,6	7,6	9,0	8,0	9,7	3,8	4,7	3,6	7,9	2-8
T	T	C	C	C	C	C	C	E	E	E	C	L L

b) Modified Proctor, Method D

Maximum density: 142,0 and 142,5* lb/ft^3 (labo).....139,0
Optimum water content: 4,8 and 5,4* % (labo).....7,0
Specific gravity: 2,69

c) CBR Load Bearing Capacity (see figure 4.3)

Test load 15,0
Density (lb/ft^3)
. before test 139,9, 139,1
. after test 139,9, 138,0
Water content (%)
. before test .....7,0 and 7,0
. after test .....5,9 and 5,6
Swelling 0,01
CBR load bearing capacity (%) .....47,4 and 70,9
Estimated Dynamic Modulus (psi) .....71100 and 106350

4.2.2.4 Characteristics of Concrete for Curbs

a) Strength (MPa)

7 days 29,4
28 days 35,5

b) Slump

Sample	Specs
90 mm (90)	80 ± 20 mm

c) Air (%)

Sample	Specs
6,6 (6,5)	4 to 7%

4.2.2.5 No-slump Concrete Characteristics

a) Mix Quantities

Cement, type 10 (lb)	500
Concrete sand (lb)	1 440
Stone 1/4" (lb)	605
Stone 1/2" (lb)	1 010
Stone 3/4" (lb)	405
Water (lb)	170
Total (lb/y^3)	4 030
Darex (ounce)	4
Slump (mm)	5
Unit weight (lb/ft^3)	153

b) Characteristics

	Number	Average	Stand. Deviation
Compressive strength (MPa)	4	38,4	33,8-42,3
Unit weight (lb/ft^3)	25	149,9	1,6
Air content (%)	25	4,9	0,2
Degree of compaction (%)	25	98,9	1,1

4.2.2.6 Bedding Sand Characteristics

a) Grading Analysis

Sieve Size	Opening	1	2	3
3/4"	19,0 mm	100,0	100,0	100,0
#4	4,75 mm	95,0	96,0	96,0
#8	2,36 mm	92,0	92,0	93,0
#16	1,18 mm	86,0	84,0	86,0
#30	600 µm	61,0	57,0	60,0
#50	300 µm	24,0	20,0	23,0
#100	150 µm	1,5	1,8	1,9

b) Proctor Density (ASTM D-698, Method D)

Unit weight (lb/ft^3)	114,8
Optimum water content (%)	12,7

c) CBR Load Bearing Capacity (see figure 4.3)

Surcharge (lb)	15,0

Unit weight (lb/ft3)	114,1 and 114,6
Water content (%)	
. before test	12,5
. after test	12,9
Swelling (%)	0
CBR load bearing capacity (%)	25,9 and 20,5
Estimated Dynamic Modulus (psi)	38 850 and 30 750

4.2.2.7 Bituminous Mix Characteristics

a) Grading Analysis

Sieve Size	MB-1	MB-2	MB-4
25,00 mm	99,0		
19,00 mm	87.0	100,0	100,0
12,50 mm	66,0	92,0	92,0
9,50 mm	53,0	75,0	82,0
4,75 mm	39,0	45,0	65,0
2,36 mm	33,0	37,0	56,0
1,18 mm	28,0	32,0	50,0
600 µm	22,0	25,0	40,0
300 µm	12,0	14,0	20,0
150 µm	6,0	6,0	8,0
75 µm	3,0	2,9	3,5

b) Characteristics

Bitumen content (%)	4,0	4,95	5,0
Voids content (%)	5,0	3,10	3,7
Voids content in aggregate (%)	14,8	15,30	15,6
Stability (MPa)	11,4	11,50	9,2
Creep (mm)	3,5	3,30	2,5
Thickness (mm)	51,9	78,90	48,3
Degree of compaction (%)	94,7	96,90	91,6

c) Degree of compaction of layers

	Tests	Average	Stand. dev.
Foundation layer (MB-1)	12,0	91,70	1,08
Binder layer (MB-2)	12,0	92,20	1,77
Surface layer	12,0	92,70	1,27

4.3 Structural evaluation during construction

To evaluate the pavement structure, each layer of pavement was studies during the course of construction. The structural characteristics investigated were the elastic moduli and other pertinent parameters determined by the "Dynalfect" dynamic deflection test.

4.3.1 Description of the "Dynaflect" test

The equipment consists of a small two-wheeled trailer housing a strong dynamic generator that applies vertical sinusoidal impulses to the structure under study using rigid wheels, 40 cm in diameter at a spacing of 51 cm. The dynamic force of 4 450 N (1 000 lb) applied at a frequency of 8 Hz adds to the static weight of 7 120 N (1 600 lb) of the assembly.

Measurements are made of the maximum deflections S_1 between the wheels and at four points aligned at 30,5 cm (12") intervals, S_2, S_3, S_4 and S_5 with velocity sensors (geophones), as shown in figure 4.4 . These measurements give the profile of the field of deflection (see figure 4.5) and make it possible to determine the indicating parameters of the state of the roas. These parameters are the elastic moduli of the different layers, the maximum deflection, the indexes SCI and BCI that characterize the curve of the test bowl and the capacity of the road to distribute the load. The significance of each of these parameters and how they may be estimated are shown in the following section.

4.3.2 Significance of Parameters Obtained from the Dynaflect Test

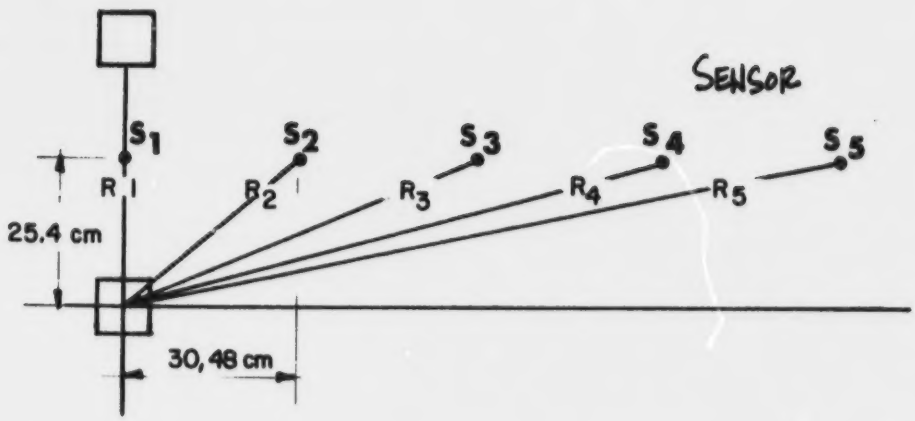
4.3.2.1 Elastic Moduli of Materials

Elastic moduli are the most important characteristics of the different pavement layers required for rational methods of road structural analysis. They are used in conjunction with the Poisson's ratio for the different materials to determine the layer thicknesses needed to meet the road service requirements during their anticipated lifespan.

Elastic moduli are determined by calculation using the method proposed by the "Texas Highway Department". The road structure in figure 4.6 : the pavement (element 1) thickness h, and the support platform with a semi-infinite thickness (element 2). Considering the Dynaflect wheel-loads applied to the structure, the pertinent parameters are (figures 4.4 , 4.5 and 4.6):

FIGURE 4.4

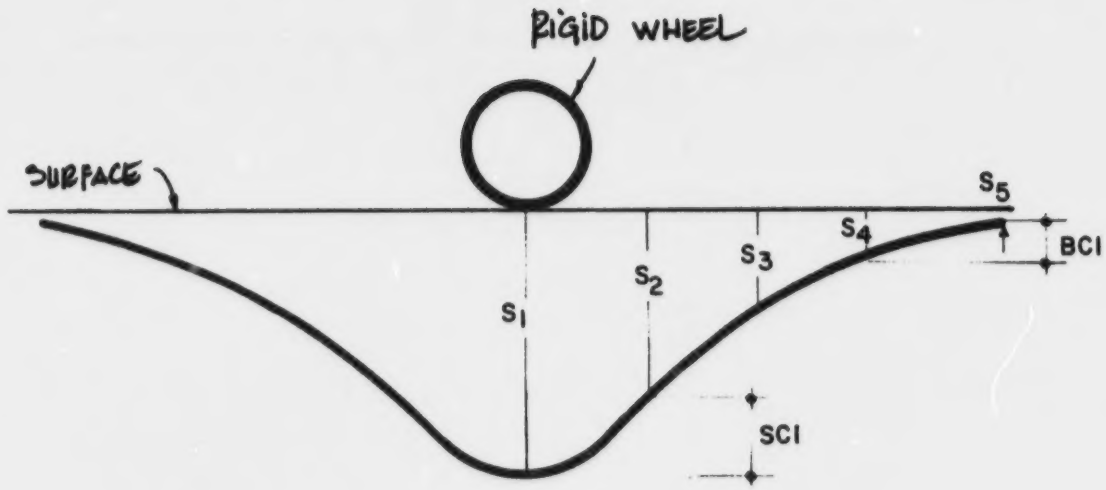
DYNAFLECT LOADING POINTS



R_1 : 25,40 cm
R_2 : 39,67 cm
R_3 :66,04 cm
R_4 : 94,74 cm
R_5 ;124,52 cm

FIGURE 4.5

DEFLECTION BOWL UNDER DYNAFLECT LOADING



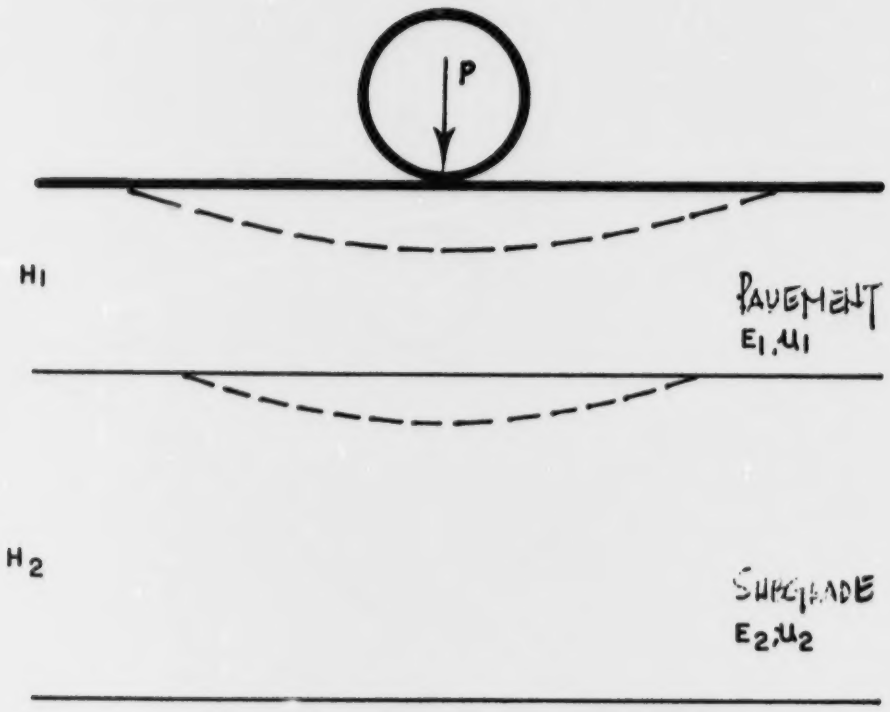
S_1 : MAXIMUM DEFLECTION

SCI : INDEX OF SURFACE CURVATURE

BCI : INDEX OF CURVATURE OF LOWER LAYERS

FIGURE 4.6

A TWO LAYER SYSTEM



P ; 4 450 N (1 000 lb) vertical load applied to a point at the surface of the two layered system.

h_1 ; thickness of the pavement.

E_1 ; elastic modulus of the pavement.

E_2 ; elastic modulus of the lower support layer.

Si ; deflection at point 1.1 inside the deflection bowl.

r_i ; distance from the loading point to the measuring point Si:

r_1 = 25,40 cm
r_2 = 39,67 cm
r_3 = 66,04 cm
r_4 = 94,74 cm
r_5 =124,53 cm

μ_1 μ_2; Poisson's ratio for layers 1 and 2.

The surface deflection (S) produced in an elastic double-layered system at a horizontal distance (r) from the applied load (P), is expressed by Burmister in the following equation:

$$\frac{4\ \pi\ E_1}{3P}\ Sr = \int_{x=0}^{x=} Vo\ Jo\ (x)\ dx \qquad (1)$$

where

x ; m r/h

m ; a parameter

Jo (x) ; Zeroeth order Bessel function, first type, argument x

$$V = \frac{1 + 4N\ m\ e^{-2m} - N^2\ e^{-4m}}{1 - 2\ N\ (1 + 2m^2)e^{-2m} + N^2 e^{-4m}}$$

and

$$N = \frac{1 - E_1/E_2}{1 + E_1/E_2} = \frac{E_2 - E_1}{E_1 - E_2}$$

Taking into account the limiting conditions of the system (limit of practical variations of x, m and E_1/E_2), the general deflection equation (1) can be rewritten as:

$$x = 10r/h$$

$$\frac{4\ \pi\ E_1}{3P}\ Sr = 1 + \int_{x=0} (V - 1)\ Jo\ (x)\ dx$$

or

$$\frac{4\ \pi\ E}{3P}\ Sr = F(E_2/E_1,\ r_1/h) \qquad (2)$$

Integration of the Bessel function is accomplished by a computerized numerical method. Considering the deflections S_1, and S_2 at points (1) and (2) at respective distances r_1 and r_2 from the loading point (figure 4.5), gives the following set of equations:

$$\frac{4\pi E_1}{3P} S_1 r_1 = F(E_2/E_1, r_1/h) \quad (3a)$$

$$\frac{4\pi E_1}{3P} S_2 r_2 = F(E_2/E_1, r_2/h) \quad (3b)$$

Dividing equation (3a) by (3b) gives:

$$\frac{S_1 r_1}{S_2 r_2} = \frac{F(E_2/E_1, r_1/h)}{F(E_2/E_1, r_2/h)} \quad (4)$$

The only unknown is the ratio E_2/E_1. By trial and error, the value of E_2/E_1 is obtained for the level of precision desired. E_1 can be obtained from equation (3a) and E_2 can then be found from the value E_2/E_1. The complete calculation process is incorporated in a computer program which also contains the relevent characteristics of the Dynaflect test load conditions (load P, distance r_i, Poisson's ratio μ_i for the two layers, the contact pressure between the pavement and wheels). The computer program receives the values for deflection S_i and the thickness h of the pavement. The calculated parameters are the elastic moduli, E_1 and E_2 (or Ep and Es), of the pavement and soil respectively, as well as the indexes described below.

4.3.2.2 Maximum deflection S_1

The maximum measured deflection is an indication of the general state of the road. It is generally considered to be an indicating parameter of the state of the support soil. But it is sensitive to both base and surface layers and should not be used alone to judge the type of weakness or the type of cracking. Values of S_1 exceeding 1,0 usually indicate a problem.

4.3.2.3 Surface curvature Index "SCI"

The surface curvature index is an indicator of surface layer structural properties. The value of SCI is inversely proportion to the radius of curvature of the deflection bowl and directly proportional to forces and tensile deformations developed at the base of the layer bounded to the surface. This parameter is defined as being the difference of deflections S_1 and S_2: $SCI = S_1 - S_2$.

4.3.2.4 Base Curvature Index "BCI"

This parameter represents the support soil and the pavement base layer conditions; it is sensitive to the soil modulus. The base curvative index is calculated as BCI = $S_4 - S_5$. Pavements having a BCI between 0,05 and 0, 11 usually behave well. When BCI is greater than 0,15, failure is assumed in the soil and/or in the base layer.

4.3.2.5 Spreadability Index "SI"

The spreadability index indicates the ability of the pavement to distribute deflections and, as a consequence, the loads. It is defined by

$$SI\ \% = i = 5 \quad \frac{i=5}{i=1} \frac{Si}{5S_1} \times 100\%$$

The higher the spreadability index, the better the pavement can distribute loads and reduce forces and deformations at the soil level. A range of SI values between 45% and 75% is usually observed for flexible pavements, and from 65% and 80% for rigid pavements.

4.4 Analyses and Strength Results of the Different Layers

The structural evaluation tests were done at different stages of pavement construction, as tables 4.1 and 4.2 indicate,to enable study of the influence of subsequent layers on the existing system characteristics. All deflection measurements and calculated parameter values are given in the appendix to this chapter.

Table 4.3 summarizes the average values of results obtained for all test road sections. The analysis and discussion of the results will refer to this table.

4.4.1 Elastic Moduli Variations of Aggregate Material During Construction

As described previously, the calculation method to estimate the elastic moduli assumes a pavement structure comprising two superimposed homogeneous elastic layers. In this way different structural combinations are analyzed to determine the modulus of such layers of material.

Average moduli calculated for dredging soil, embankment and sub-foundation materials (0-75 mm) are presented in figure 4.7. In all cases the magnitude of the elastic modulus of an aggregate material increases as each layer is added to the existing system. The multiplying factors for modulus are in the order of 2,0 to 2,5, as the layers under study go from the surface position to the complete pavement structure.

TABLE 4.2

ACTUAL REALIZATION OF DYNAFLECT DEFLECTION MEASUREMENTS

DURING SECTIONS OF TEST

TEST NUMBER	BLOCK PAVEMENT		ASPHALT LAYER	CONCRETE BASE	CRUSHED STONE Base and Subbase		FILL SUBGRADE	HYDRAULIC FILL
	Before	After			0-20 mm	0-75 mm		
A	X	X	-	-	X	X	X	X
B	X	X	-	-	X	X	X	X
C	-	-	X	-	X	X	X	X
D	X	X	-	X	-	X	X	X
E	X	X	-	X	-	X	X	X

X: Dynaflect measurement done

-: No Dynaflect measurement

TABLE 4.3

SUMMARY OF DEFLECTION READINGS

AVERAGES FOR TEST PAVEMENTS

	Elevation of measurement	Test sections DE	C	AB	Dynaflect deflection readings S_1	S_2	S_3	S_4	S_5	SCI	BCI	SI	Elastic modulus	
	On block pavements												E(pavement)	E(subgrade)
A	After compaction			X	0.86	0.35	0.20	0.13	0.09	0.51	0.04	38	21,800	62,500
A	Before compaction				0.67	0.31	0.18	0.13	0.09	0.37	0.03	41	30,050	64,450
B	After compaction			X	0.88	0.35	0.19	0.13	0.09	0.47	0.04	39	23,200	62,600
B	Before compaction				0.66	0.29	0.15	0.10	0.07	0.37	0.03	38	30,100	67,538
													E(pavement)	E(subgrade)
D	After compaction	X			0.24	0.12	0.11	0.09	0.08	0.12	0.02	56		
D	Before compaction				0.26	0.10	0.11	0.10	0.08	0.16	0.01	69	510,450	85,100
													E(pavement)	E(subgrade)
E	After compaction	X			0.24	0.14	0.12	0.11	0.09	0.11	0.02	62		
E	Before compaction				0.21	0.08	0.08	0.08	0.07	0.13	0.01	60	480,350	74,200
	Asphalt course		X		0.61	0.40	0.25	0.15	0.10	0.21	0.05	50	E(asph.conc) 264,500	E(subbase) 40,000 / E(subgrade) 36,000
	Concrete base	X			0.17	0.16	0.15	0.13	0.12	0.01	0.02	87	E(concrete) 8,700,000	E(subgrade) 35,250
	0-20 On base		X	X	0.76	0.37	0.23	0.16	0.12	0.39	0.04	43	E(0-20) 27,317	E(subgrade) 43,900
	0-75 On subbase	X	X	X	0.77	0.43	0.29	0.24	0.20	0.35	0.04	50	E(0-75) 31,100	E(subgrade) 31,900
	Fill subgrade	X	X	X	0.93	0.58	0.38	0.29	0.23	0.35	0.06	52	Er(0-40") 28,000	Es(40"+) 24,750 n=55
	Hydraulic fill	X	X	X	1.76	92	44	27	18	84	0.09	41	Es(0-24") 11,650	Es(24"+) 20,400 n=25

TABLE 4.3

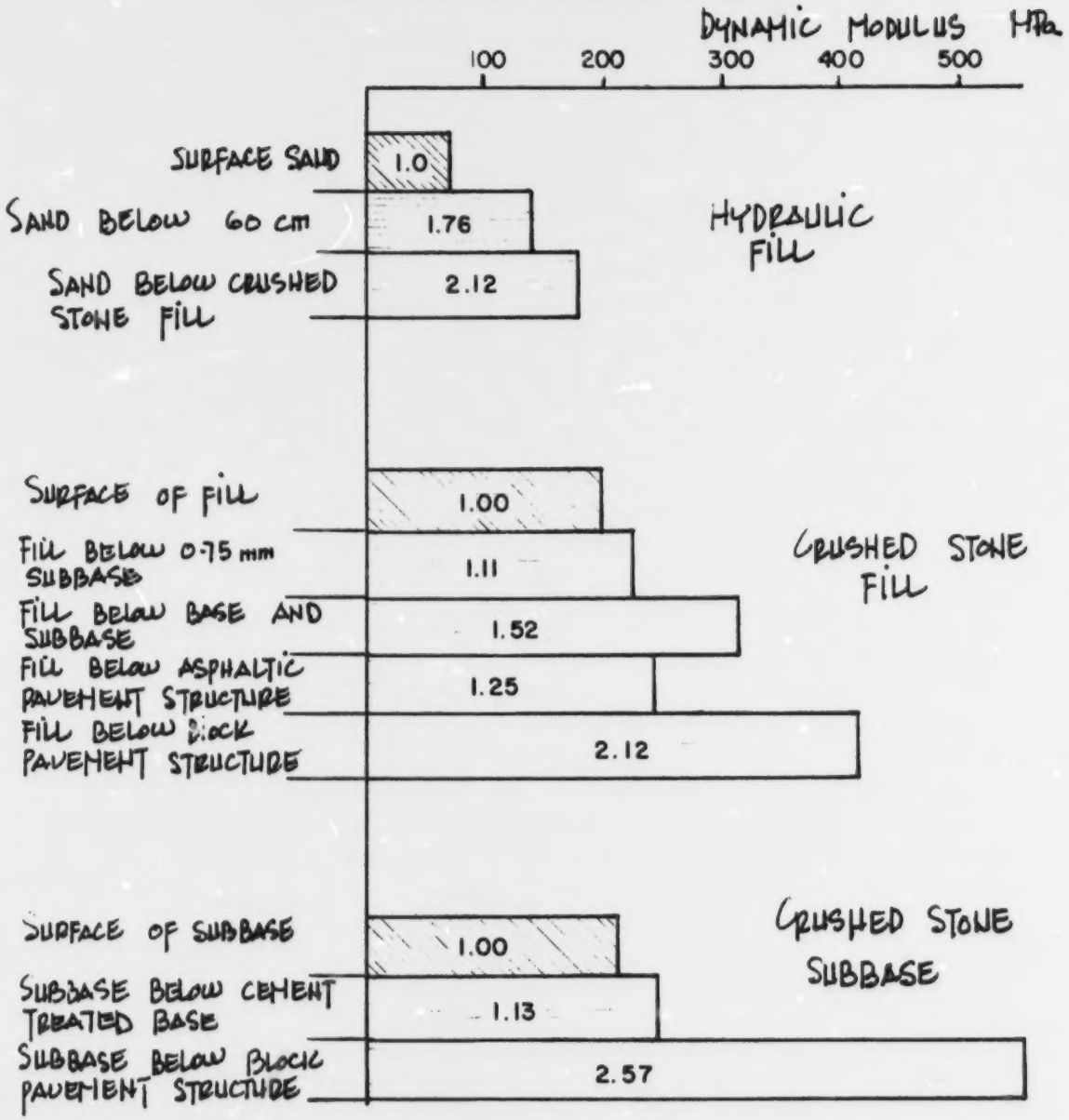
SUMMARY OF DELFECTION READINGS

AVERAGES FOR TEST PAVEMENTS

Elevation of measurement		Test sections			Dynaflect deflection readings					SCI	BCI	SI	Elastic modulus		
		DE	C	AB	S_1	S_2	S_3	S_4	S_5						
	On block pavements														
A	After compaction			X	0.86	0.35	0.20	0.13	0.09	0.51	0.04	38	E(pavement) 21,800	E(subgrade) 62,500	
A	Before compaction				0.67	0.31	0.18	0.13	0.09	0.37	0.03	41	30,050	64,450	
B	After compaction			X	0.88	0.35	0.19	0.13	0.09	0.47	0.04	39	23,200	62,600	
B	Before compaction				0.66	0.29	0.15	0.10	0.07	0.37	0.03	38	30,100	67,538	
D	After compaction	X			0.24	0.12	0.11	0.09	0.08	0.12	0.02	56	E(pavement)	E(subgrade)	
D	Before compaction				0.26	0.10	0.11	0.10	0.08	0.16	0.01	69	510,450	85,100	
E	After compaction	X			0.24	0.14	0.12	0.11	0.09	0.11	0.02	62	E(pavement)	E(subgrade)	
E	Before compaction				0.21	0.08	0.08	0.08	0.07	0.13	0.01	60	480,350	74,200	
Asphalt course			X		0.61	0.40	0.25	0.15	0.10	0.21	0.05	50	E(asph.conc) 264,500	E(subbase) 40,000	E(subgrade) 36,000
Concrete base		X			0.17	0.16	0.15	0.13	0.12	0.01	0.02	87	E(concrete) 8,700,000		E(subgrade) 35,250
0-20 On base			X	X	0.76	0.37	0.23	0.16	0.12	0.39	0.04	43	E(0-20) 27,317		E(subgrade) 43,900
0-75 On subbase		X	X	X	0.77	0.43	0.29	0.24	0.20	0.35	0.04	50	E(0-75) 31,100		E(subgrade) 31,900
Fill subgrade		X	X	X	0.93	0.58	0.38	0.29	0.23	0.35	0.06	52	Er(0-40") 28,000		Es(40"+) 24,750 n=55
Hydraulic fill		X	X	X	1.76	92	44	27	18	84	0.09	41	Es(0-24") 11,650		Es(24"+) 20,400 n=25

FIGURE 4.7

INFLUENCE OF SURCHARGES ON RESILIENT MODULI OF GRANULAR LAYERS



This increase in aggregate modulus with pavement thickness shows, in fact, it is dependent on the stress level to which the material is subjected. We know that the soil and aggregate modulus E_2 is related to the stress level ty the equation:

$$E = K\sigma_d^n$$

where

K ; constant.

n ; stress dependant parameter.

σd ; $\sigma 1 - \sigma 3$ difference in stress level

$\sigma 1$; axial compressive stress (vertical)

$\sigma 3$; lateral stress

In proceeding from the surface to the lower layers, a given aggregate layer is confined by adjacent layers, and this confining effect is increased with each additional layer of the structure. The result is an increase in the vertical compressive stress in the material, $\sigma 1$, and consequently also σd and the modulus E_2.

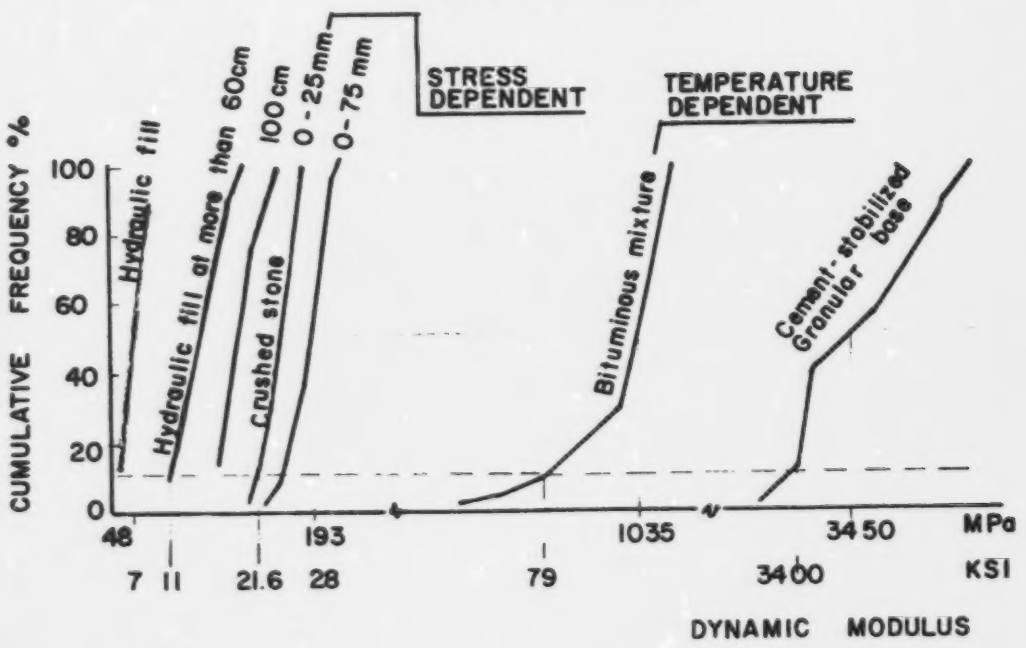
Determination of the moduli as they appear in the results is not always successful. This was the case especially for the stone layer (0-20 mm) when considered alone at the surface. That is why the influence of upper layers on this material are not analyzed. However, one expects lesser or, at most similar effects to those which are taken for other aggregate layers since the 0-20 mm stone supports only the surface layer.

The range of estimated average dynamic moduli for different materials and systems used in the test program and the observed standard deviations (σ) are presented below:

Materials	Nb of tests	Dynamic Modulus ($x10^3$ psi)	Standard Deviation
Bituminous surface layer	48	265	2
Concrete base	48	8 700	4 100
Concrete pavement	48	500	1 300
Pavement on aggregate	48	22	4
Crushed stone (0-20 mm)	48	40	2

FIGURE 48

CUMULATIVE FREQUENCY CURVES OF DYNAMIC MODULUS OF PAVING MATERIALS



	Nb of tests	Dynamic Modulus ($x10^3$ psi)	Standard Deviation
Crushed stone (0-74 mm	50	31-85	4-15
Embankment	55	29-63	7-11
Dredgning soil	25	12-25	6-7

The method used to determine the moduli assumes a double-layered system while the structures analyzed usually contain several layers of material. The moduli obtained often relate to an ensemble of materials. Certain techniques are proposed to estimate the elastic modulus of each layer but these empirical techniques have the drawback of not taking into account a given variation in the layer modulus with that of the structure of which it is a part. It is well known that the elastic modulus of an aggregate material bears a direct relationship to the stress to which the material has been subjected.

It is for this reason that moduli obtained for heterogeneous material layers must be associated with the corresponding systems under study to be significant. Furthermore, the elastic modulus of a given material is not a constant and varies from one location in the road to another. In figure 4.8 , the elastic moduli frequency curves are illustrated for surface soil, internal soil, embankment, stone 0-20 mm, stone 0-75 mm, bituminous surface layer and portland cement concrete.

4.4.2 Influence of Different Layers on the Structural Characteristics of a Pavement

Figures 4.9, 4.10, 4.11 represent variations in deflection bowls, the spreadability index and maximum deflection values during construction, respectively. They make it possible to analyze effects caused by subsequent layers on the structural capacity of the test pavements.

As figure 4,11 illustrates when dredging soil was the first support element, it created the largest deflections (S_1, S_2, S_3) on sensors 1, 2 and 3. It is evident that this semi-infinite layer of non-compacted soil is not stable and is barely able to distribute the loads. The poor surface soil compaction is indicated by a very high value of the "Surface Curvature Index", 0,84, and a Spreadability Index of only 41%. However, the base curvature index of below-grade soil at 0,09 is quite satisfactory since the Index limit for a good soil is 0,11. The best compaction for the below-grade soil is also shown in the improvement of the elastic modulus by a factor of 1,76, as illustrated in the diagram of figure 4.7.

FIGURE 4.9

DYNAFLECT DEFLENION BOWL FOR TEST SECTIONS

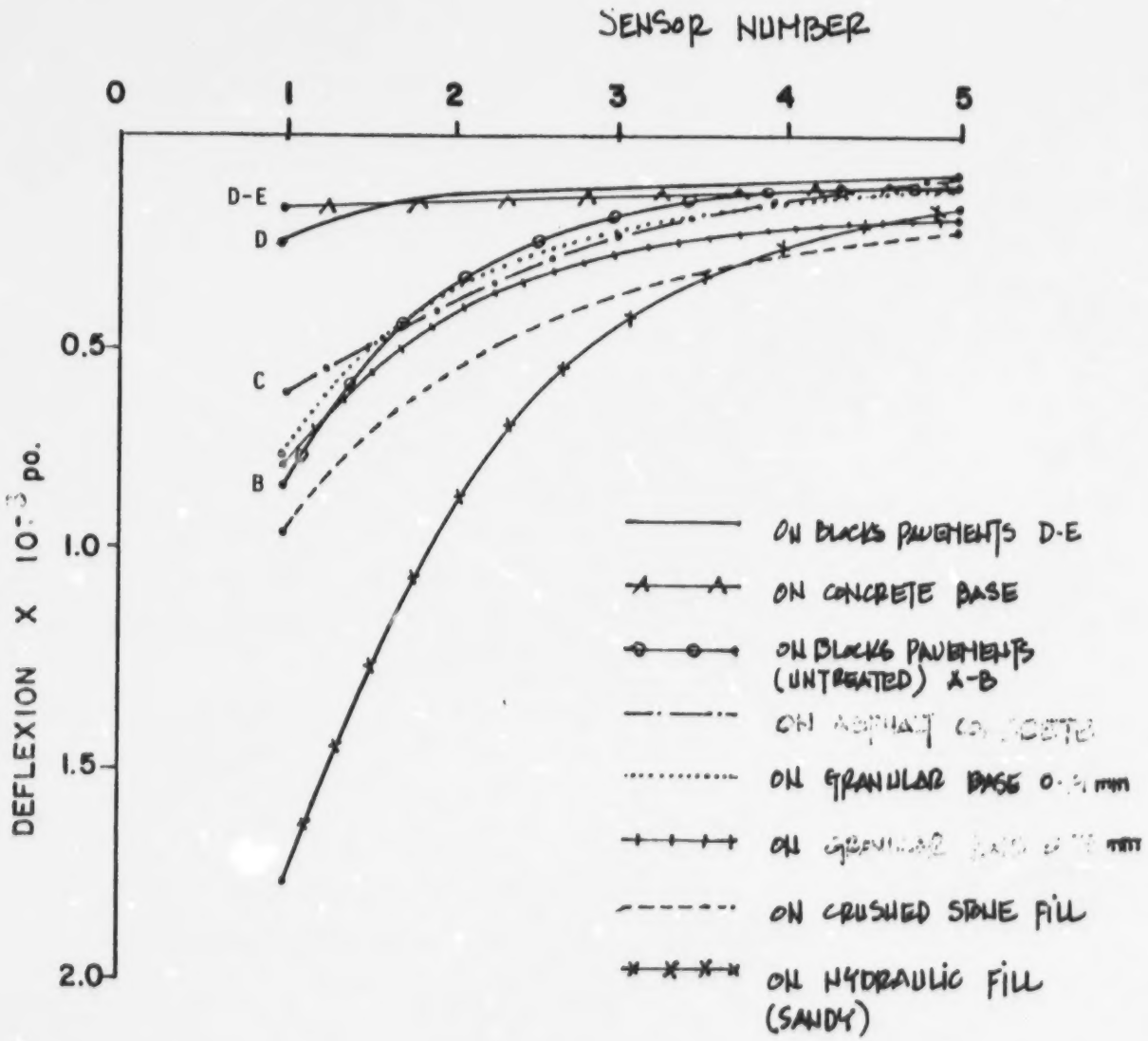


FIGURE 4.10

VARIATION IN SPREADABILITY DURING CONSTRUCTION

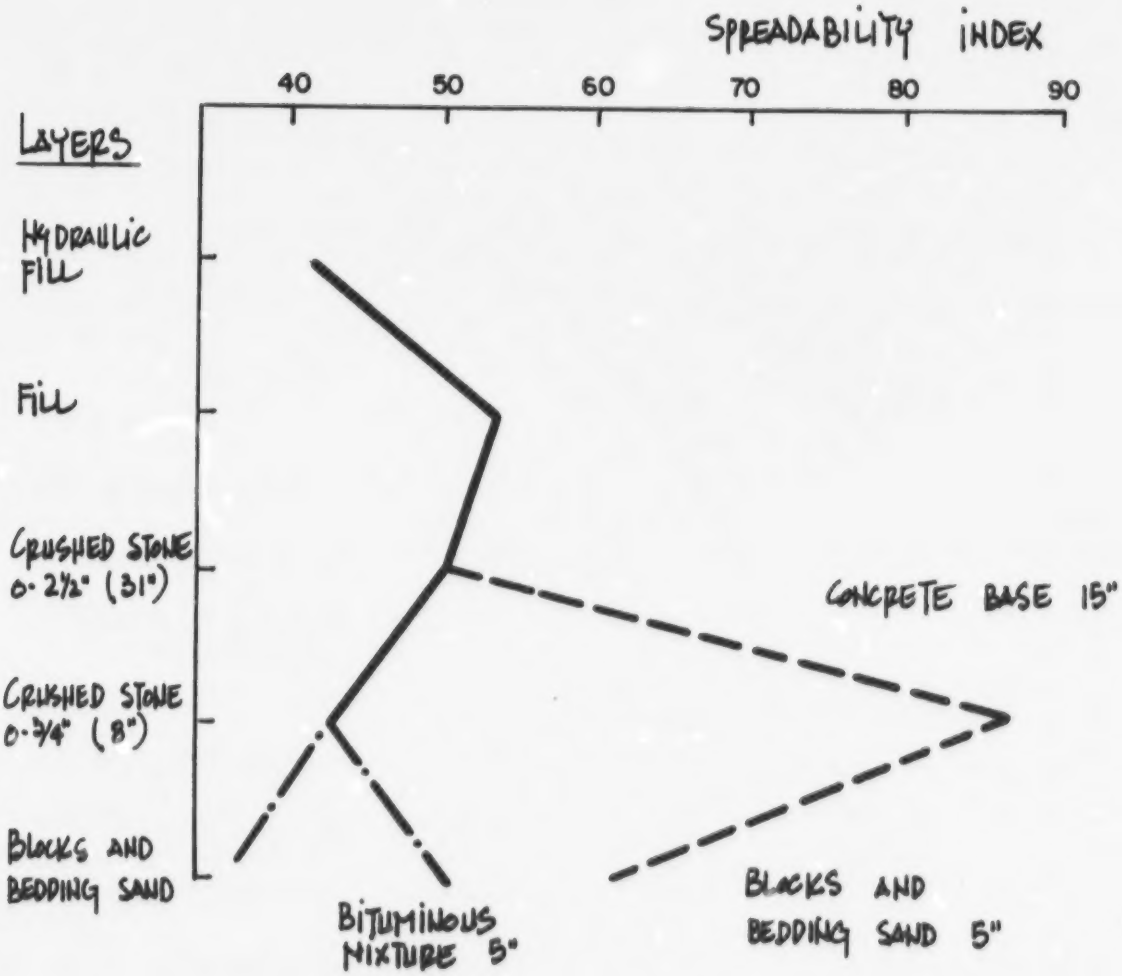
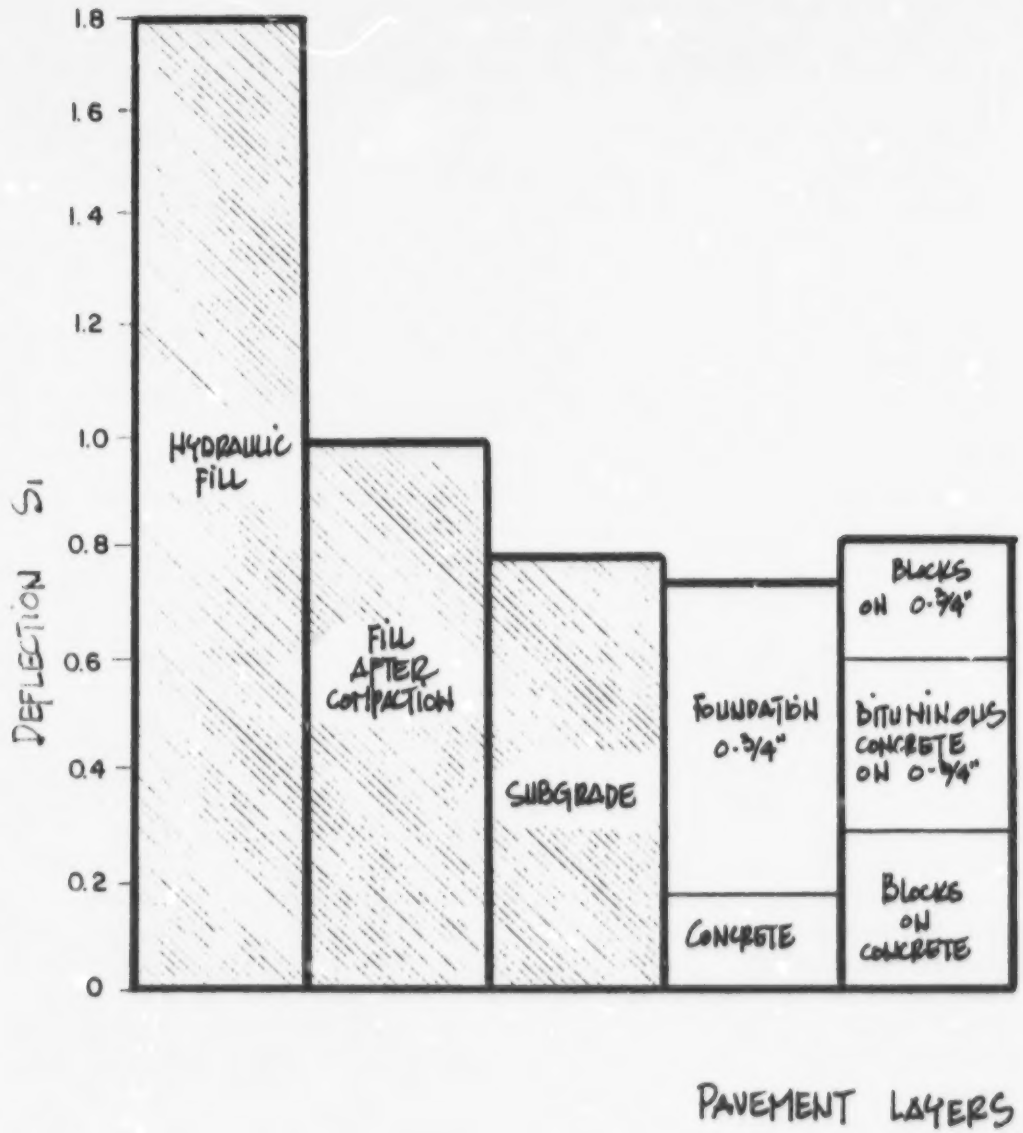


FIGURE 4.11

VARIATION IN DEFLECTION OF DIFFERENT LAYERS



4.4.3 Influence of Aggregate Layers on the Structural Capacity

4.4.3.1 Influence of Layer of Stone Fill on Dredging Soil

Adding a layer of dynamically compacted stone fill caused substantial reductions in deflections S_1 and S_2 of 47 and 37 per cent, although it did not have much influence on the other deflections. This induced a net decrease in the Surface Curvature Index from 0,84 to 0,35. The capacity of the new structure to spread the load was also improved, and the spreadability index rose from 11 to 52%.

The results illustrate well the value of a layer of well-compacted stone fill having a dynamic modulus about 30 000 to substantially improve the structural capacity of the pavement. The compaction and weight of the fill improved the quality of the dredging soil where the elastic modulus increased from 11 650 on the surface to 24 800 psi under the fill. The curvature index also decreased by 33%.

4.4.3.2 Influence of a Layer of 0-75 mm stone resting on stone fill

The new deflection bowl of the 0-75 mm crushed stone layer is almost identical to that observed on the fill. The deflections S_1 and S_2 are reduced to the same extent with the new layer, resulting in an identical surface curvature index of 0,35 for the systems with and without a stone layer. The addition of a 0-75 mm crushed stone layer does not, therefore, improve the road surface quality.

Incidentally, the structural role is apparent in the base curvature index being reduced from 0.06 to 0.04.

4.4.3.3 Influence of a 0-20 mm Crushed Stone Layer Resting on 0-75 mm Stone

The overall quality indicator S_1 remained unchanged with the addition of 0-20 mm stone. One observes, however, appreciable reductions for other deflection parameters: notably the surface curvature index which increased from 0,35 to 0,39 and suggests that the 0-20 mm crushed stone is less good structurally then the 0-75 mm stone. Furthermore, there is a reduced capability of the structure to distribute stresses, with the spreadability index going from 52% to 43%. This layer, used mostly to level and even out the surface of the 0-75 mm layer offers little to improve the load bearing capability of the road except when it is confined under a wearing surface.

4.4.4 Influence of Upper layers on the Structural capacity

4.4.4.1 Influence of a Concrete Base

The influence of the compacted concrete layer resting on a platform of dredging soil fill and 0-75 mm aggregate subfoundation is even more remarkable: the maximum deflection decreased by 80%; the spreadability index increased from 50% to 87%; the surface curvature index (SCI) having decreased 97% to a value of 0,01; and the base survature index fell by 50% to 0,02.

As expected, the large rigidity of no-slump concrete and its significant thickness made it possible to attain a spreadability index exceeding the 65 to 80 percent of conventional rigid roads.

4.4.4.2 Influence of Paving Blocks Resting on a Concrete Base

The discussion which follows will apply to results obtained after compaction of the paving block bedding sand finishing layer.

No appreciable difference appears between the parameter values obtained for roads D and E. For these two roads, the maximum deflection S_1 is greater than that of the concrete while the opposite holds for the other deflection S_2 and S_5, which are reduced. The surface curvature index increases from 0,01 to 0,12 but the BCI remains unchanged.

This indication of a weakening in the road surface by the increase in the SCI is understandable: the structure of concrete, bedding sand and paving blocks cannot have the same stability as that of a monolithic concrete base no matter what level of compaction was obtained. Note that the capability of the structure to spread the load also decreased with the placement of paving blocks: the spreadability index fell from 87% before blocks were laid to 56% and 62% after laying.

4.4.4.3 Influence of Paving Blocks Resting on 0-20 mm Crushed Stone Base

The small differences in thickness between roads A and B are not apparent from the results. The maximum deflection is greater for paving blocks than for the 0-20 mm stone while all the other deflections at S_2 and S_5 are reduced. The Base Curvature (BCI) did not change, but the Surface Curvature Index (SCI) increased from 0,39 to 0,47 and 0,51.

Here again, the weakness appearing in the road surface under light loads is attributable to the instability of the system comprised of base layer, bedding sand and paving block. This system has the same effect on the road as a layer of granular material (see figure 4.10). Its beneficial effects on the load bearing properties of the system increases with an increases in heavy traffic flow during the use of the Terminal.

4.4.4.4 Influence of Bituminous Surfacing on a 0-20 mm Stone Base

The layer of bituminous mix reduced the maximum deflection S_1 by 20%, while the other deflections barely changed from their initial values before surfacing. The result was an appreciable reduction in the surface curvature index, from 0,39 to 0,25 and a consequent increase in the spreadability index to 50% compared to 43% on the foundation. The BCI value remained unchanged.

4.5 Influence of Roller Compaction on Block Pavement

The influence of compaction on the block pavement, using a 30-ton roller, was determined by level measurements and by deflection tests before and after compaction. The results are summarized in tables 4.4 and 4.5 and reveal that compaction:

a) had little influence on the packing of the block pavement surface;

b) did not affect the structural capacity of the roads with a concrete base (Table 4.5a);

c) had a harmful effect on the strength of roads with an aggregate base, notably by (Table 4.5b):

- increasing S_1 (an overall weakening of the road)
- increasing SCI (weakening of the surface)

Note that for block pavements the structural capacity increases with an increase in the interlocking effect of the paving. This suggested the possibility that with time the operation of heavy vehicles and warehouse stock piling will contribute to enhancing the load distributing capacity of the block pavement.

TABLE 4.4

INFLUENCE RESULTING FROM 5 PASS OF AN RUBBER TYRED 30 T. COMPACTOR ON LEVELLING OF BLOCKS PAVEMENT RESTING ON A CEMENT TREATED BASE.

LIGNE 0+40

STATION	AFTER COMPACTION	BEFORE COMPACTION	DIFFERENCE (mm)
0 + 15	1.812	1.812	0
0 + 20	1.838	1.838	0
0 + 25	1.858	1.859	- 1
0 + 30	1.872	1.872	0
0 + 35	1.878	1.880	- 2
0 + 40	1.869	1.869	0

LIGNE 0+44

STATION	AFTER COMPACTION	BEFORE COMPACTION	DIFFERENCE (mm)
0 + 15	1.750	1.749	+ 1
0 + 20	1.767	1.767	0
0 + 25	1.778	1.778	0
0 + 30	1.811	1.812	- 1
0 + 35	1.823	1.822	+ 1
0 + 40	1.838	1.840	- 2

REF. : PORT OF MONTREAL RESULTS

TABLE 4.5a

INFLUENCE OF ROLLING WITH A 30 T. TYRED COMPACTOR ON DEFLECTION OF A BLOCK PAVEMENT (100 mm blocks) RESTING ON A CEMENT TREATED BASE (SECTION E)

a) Dynaflect deflection datas

	BEFORE COMPACTION		AFTER COMPACTION	
	$\bar{X}$	σ	$\bar{X}$	σ
Maximum deflection 10^{-3}mm	5.3	2.2	6.1	3.3
SCI 10^{-3} mm	3.3	2.2	2.79	3.3
BCI 10^{-3} mm	0.2	0.2	0.5	-
SI %	60	34	62	12

b) Elastic Moduli calculation (Dynaflect loading)

	BEFORE COMPACTION	AFTER COMPACTION
Elastic Moduli (Mpa)		
Pavement	5250	3310
Supporting layer	766	510

TABLE 4.5b

INFLUENCE OF ROLLING WITH A 30 T. TYRED COMPACTOR ON DEFLECTION OF A BLOCK PAVEMENT (100 mm) RESTING ON AN UNTREATED GRANULAR BASE (TEST SECTION B)

a) Dynaflect deflection data

	BEFORE COMPACTION		AFTER COMPACTION	
	$\bar{x}$	σ	$\bar{x}$	σ
Maximum deflection 10^{-3}mm	18.28	2.81	20.83	3.81
SCI 10^{-3} mm	10.41	3.56	11.94	3.81
BCI 10^{-3} mm	0.76	0.25	1.02	0.25
SI %	40	6	39	3

b) Elastic Moduli calculation (Dynaflect loading)

	BEFORE COMPACTION	AFTER COMPACTION
Elastic Moduli (Mpa)		
Pavement structure	n.a.	453
Supporting layers	n.a.	160

n.a. = no unique solution in interpreting dynaflect deflection readings

5. INSTRUMENTATION OF TEST SECTIONS (1)

5.1 Purpose of Instrumentation

Physical properties of paving materials are influenced by the variation in temperature and moisture, with time. In the context of this study, it is important to clearly identify the possible variations in temperature and moisture content of the five test sections. The study of the winter behaviour of the test sections would be incomplete without the measurement of at least two important environmental parameters:

- the distribution of relative temperatures in the road profile and its variation with time: this requires the measurement of the lowest temperature reached at the surface and establishing a temperature profile between the surface and a relative point of small annual variation;

- the distribution of water in the foundation: the presence of water influences the saturation level of foundation material and, as a consequence, its mechanical properties. In the case studies, the height of the water table can be related to the river water level which was found to be 3 to 4 metres below the road. Following from this the foundation moisture conditions will be influenced by deposits at the surface (by way of joints and cracks in the pavement) and by the residual profile[1] establishing itself at some level between the water table and the surface. The profile here is also influenced by temperature variations, especially at the freezing point.

Knowing this residual profile and its variation at freezing is necessary in order to predict pavement behaviour and the effects of frost (cracking, subgrade movements, loss of bearing capacity on thawing and other structural effects related to soil saturation).

5.2 Choice and Installation of Instrumentation

Several types of apparatus are available to evaluate temperature and humidity profiles in soil located above or below the water table. Selection should be based on reliability and economics.

5.2.1 Temperature Measurements

Two types of instruments were used according to requirements: frost indicators and temperature measuring devices.

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1 Residual Profile: the theoretical water content resulting from the equilibrium established by the forces of suction and gravity on the free water present in the soil.

(1) written by M.A. Laforte, p. eng.

As the name implies, frost indicators are used especially to establish the frost line (or thawing) with time. Instrument installation is simple and readings are easily made. The instrument costs little to make, is known to be durable, and precision can reach $\pm$ 1°C.

Temperature probes measure ambient temperature at a given point. There are many devices available, such as resistance thermometers and thermocouples, depending on the desired precision and budget constraints.

The establishment of a temperature profile requires many measurement points in the same locality. With a limited budget it is important that the selected instrument be inexpensive still being reliable. Because of project requirements, (punctual recordings, large variations in local temperature), copper-constantan thermocouples were used, providing a precision of 0,1°C (0,05°F).

A "DIGIMITE" instrument, having a precision of .5°C ($\pm$ 1°F), was used for the readings.

5.2.2 Moisture measurement

As in temperature measurements, different instruments can be used to evaluate the moisture profile variations in soils that are usually unsaturated: tensionmeters, nuclear density meters, psychrometric meters. Most of these instruments are expensive but are reliable and have adequate response times. Some require preliminary calibration while others provide water contents almost direct (i.e. nuclear density meters).

The measuring device used was a "TROXLER" neutron-neutron meter as used in sounding. Using a probe the meter measures the quantity of water (in g/m^3) located in a 15 cm cylinder of soil. Measurements can be made with this device in 15 cm steps to a depth of 3 metres.

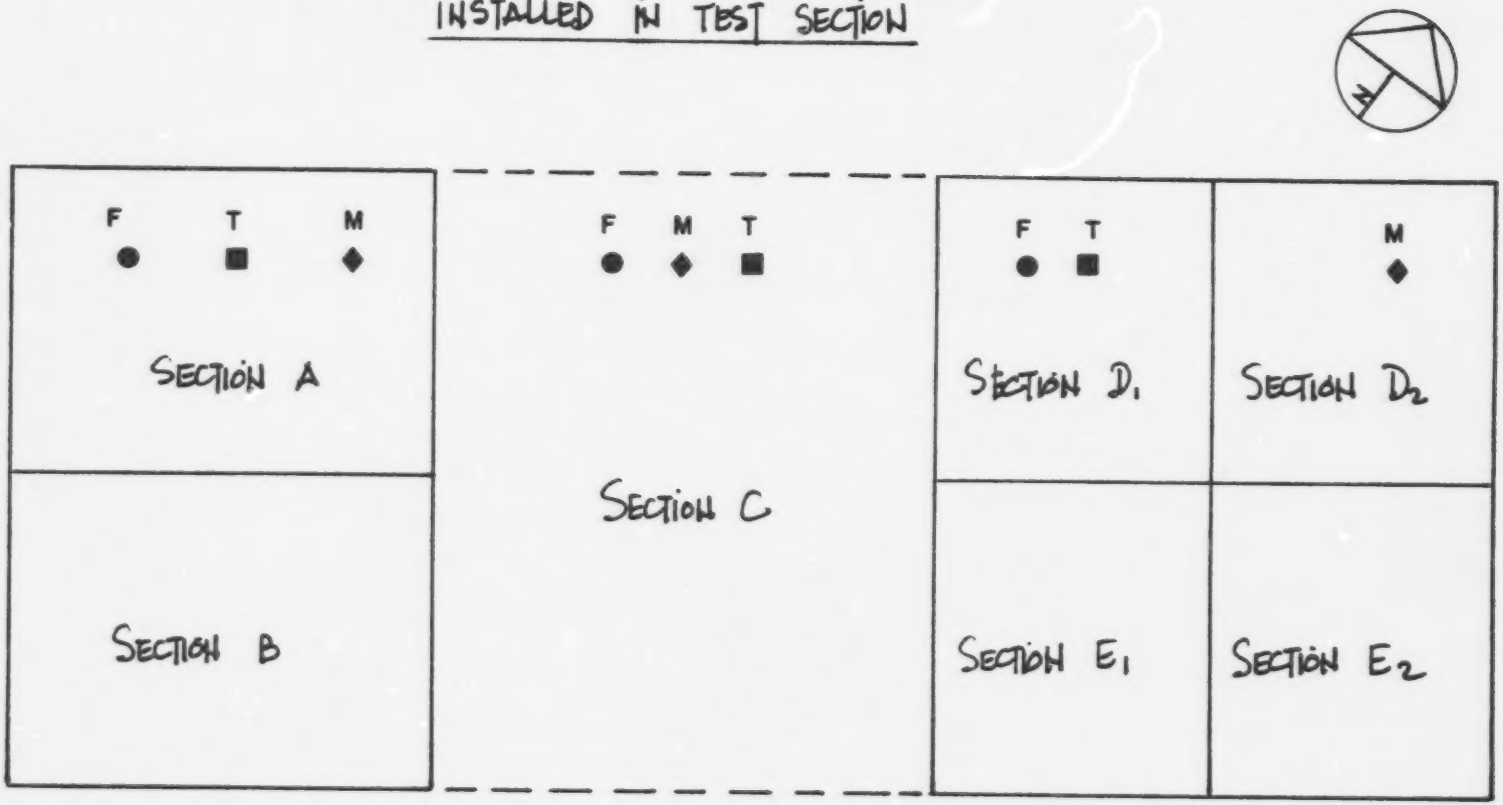
For correct use, the previously calibrated device requires a probe located at the depth to be tested. The probe is furnished with a thin aluminum tube through which the measurement is made.

5.2.3 Instrument Installation (figure 5.1)

The instruments were installed in test holes on November 20, 1981. The selected locations for instruments recognized the space required by the stored containors. In the same way, instrument preparation and final installation were made taking into consideration the paved surfaces under conditions of normal use and at times of snow removal: that is, on a horizontal level and slightly recessed below the finished surface.

FIGURE 5.1

LOCALISATION OF INTRUMENTATION INSTALLED IN TEST SECTION



F FROST TUBE INDICATOR

T TEMPERATURE MEASUREMENTS UNITS (THERMOCOUPLE)

M ACCESS TUBE FOR NEUTRON - NEUTRON MOISTURE MEASUREMENT UNIT

The holes were made by jackhammers (with a 5 cm bit) to the required depth, or by driving tubes into drilled holes. This second method was necessary in cases where the driven bits met too much soil resistance or else the holes collapsed because of soil characteristics.

The first type of drilling was chosen because it was inexpensive and fast (for drilling to depths of 5 metres, 5 to 10 m/hr, including displacement time). However, the holes used for the protective tubing of the moisture probes had to be made by driving the tubes and by rotary drilling.

The space around the instruments was filled with fine sand having the same characteristics as that sampled by drilling (above and below 1,3 m depth) or else well-compacted stone screenings (between the surface and 1,3 m of depth in general).

5.3 Detailed Description of Instruments

The frost indicators (3 units), the thermocouple sets (3 sets of 5 thermocouples) and the moisture tubes (two 1,6 m long and one 3 m long) were installed on site; that is, for comparison, one unit of each type on each test section.

5.3.1 Frost Indicators (figure 5.2)

A modified "GANDAHL" frost indicator was used. The assembly consisted of:

1. A protective exterior CPVC tube formed of 1,5 m long sections, 75 mm ID, coupled with glued external clamps. The tubing is closed at the bottom by a glued plug.

2. A "TYGON" transparent tube, 50 mm ID, containing a .005% solution of methylene blue in distilled water, was used to ensure a freezing point of 0°C. The solution changed colour (from blue to white) at the frost line, and the reading consisted of measuring the distance from the surface to the contact face of the two colours.

5.3.2 Thermocouple Units (figure 5.3)

Each measuring unit is made up of six to nine copper-constantan thermocouples with soldered junctions and secured to a 12 mm Ø wooden rod to maintain preset distances during installation in the drilled hole.

Distances between thermocouples are measured on a logarithmic base because temperature variations decrease significantly with distance from the surface.

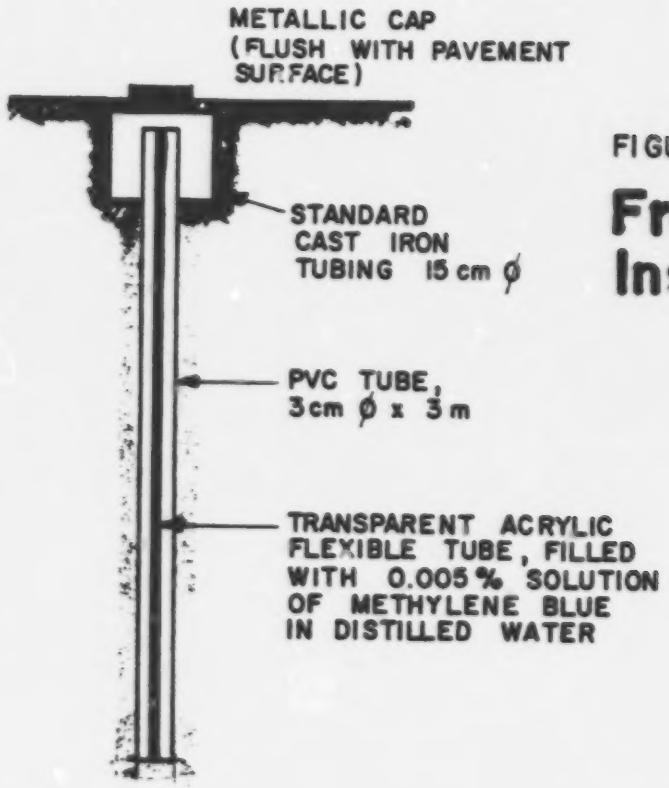


FIGURE 5.2

Frost Tube Installation

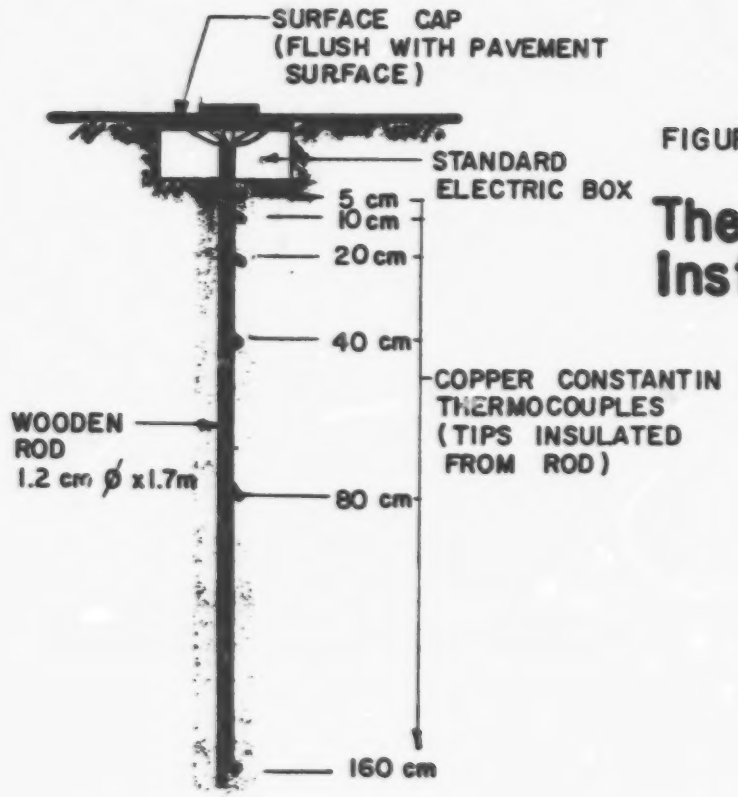


FIGURE 5.3

Thermocouple Installation

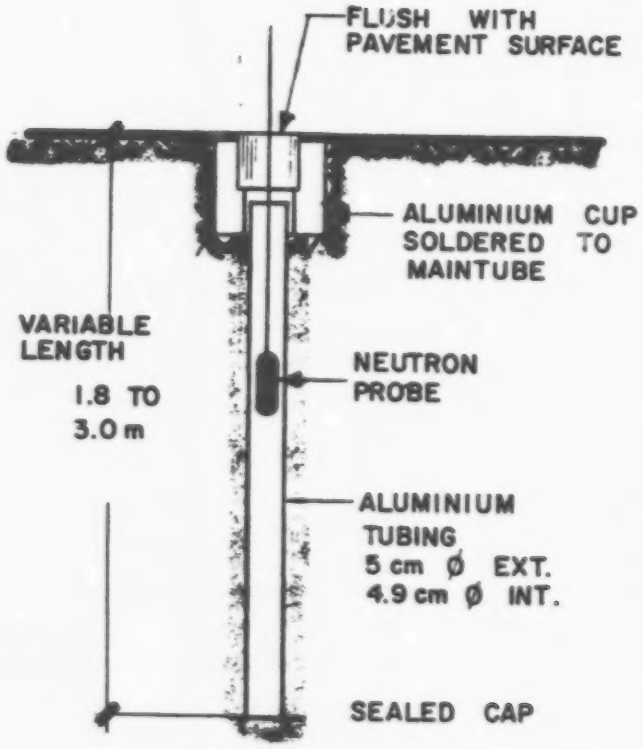


FIGURE 5.4

Access Tube for Neutron Probe

In general, thermocouples were set at depths of 160 cm, 80 cm, 40 cm, 20 cm, 10 cm and 5 cm and each 100 cm after that (i.e. at 260 cm, 360 cm) for deeper holes.

Readings were made with a "DIGIMITE" meter featuring automatic compensation and with a pre-calibrated reference junction for distances up to 300 m away. The reading precision was 0,5°C (1°F); although that of thermocouple was 0,1°F (0,05°C), it was judged unecessary for the actual study.

5.3.3 Moisture Tubes (figure 5.4)

The TROXLER device used for measuring the moisture profile requires the protective aluminum tubes to be placed at least 40 cm deeper than the measurement level desired (to accomodate the dimensions of the measurement cell. This standard tube consists of thin-walled (49 mm ID, 50 mm OD) aluminum tube, sealed and soldered at the end.

Before actual measurements are taken a dummy probe of similar dimensions is inserted to check the condition of the tube and for the presence of water or ice. This precautionary measure is necessary because of the tight fit between measuring probe and tube and helps avoids loss of a costly device or reduce the risk of an accident and radiation.

6. ECONOMIC ANALYSIS OF PAVEMENTS (1)

6.1 Introduction

The economic analysis of pavements can be approached in two ways:

- It may be based on a detailed and precise description of present and future costs and benefits, in order to determine the time-dependent flow of funds as well as the yield from the investment;

- It may also consist in an examination of the variation in cost, over time, given different strategies. That is to say, one might wish to classify design strategies by priorities, determine the constraints and thus develop the potential risks for each contemplated strategy.

This chapter is comprises of five sections:

a) Methods of economic analysis;

b) Objectives of an economic analysis;

c) Pertinent parameters;

d) Development of a computerized model; and

e) Application of the model to the test pavements at the Port of Montreal's Racine Terminal.

6.2 Methods of analysis

Methods currently used in the study of cost or costs and benefits are:

- The present value of costs;
- The present value of benefits;
- The net present value (costs/benefits);
- The uniform annual cost equivalent method;
- The internal rate of return;
- The cost/benefit ratio;
- The efficiency/cost ratio.

6.2.1 Present value of costs method

The present value of costs method determines the current costs of all expenditures incurred by a project during a given period of time. Different investment strategies are compared by determining the value of the project money with time - as would be expressed by equations for compounded interest.

(1) with Gilles Hurtubise, M.A. Laforte and, economist

6.2.2 Present value of benefits method

This method is identical to the preceding one except that benefits are measured instead of costs. Here, the major difficulty is finding an objective definition of indirect benefits.

The benefits generally include all the real advantages generated by a new investment. Often these benefits cannot be quantified or else necessitate a subjective estimation. Also, the time and effort required to determine and compute the relevant data do not always justify the additional precision gained in the results.

6.2.3 Net Present value method

The net present value of a project is obtained by deducting the present value of all costs from that of the benefits. A project is economically feasible if this difference is greater or equal to zero. Of all economically feasible projects the one with the highest net present value will be the most profitable.

6.2.4 Uniform annual cost equivalent method

This method entails computing all costs, initial and future, and then spreading them out annually in equal amounts over the time-horizon of the analysis. The merit of this procedure is that it takes into account not only the costs of a project but also compares the costs over the life of the project. This method is highly recommended when different maintenance programs are to be compared.

6.2.5 Internal rate of return

The internal rate of return is the interest rate which brings into balance the costs and benefits of a given project.

In fact, it represents the rate at which the net present value of a project equals zero. The higher the internal rate of return, the more economically feasible is the project.

6.2.6 Cost/benefit ratio

The cost/benefit ratio takes somewhat the same approach as the net present value, but rather than computing the difference between the present value of costs and that of the benefits, it utilizes the ratio of these values. With this method, a project is economically feasible if that ratio is equal to or greater than one.

6.2.7 Efficiency/cost method

The efficiency/cost method is used primarily when the different solutions offer substantial non-monetary advantages. This method necessitates a subjective evaluation of the benefits and an aptitude in judging the degree of efficiency.

6.2.8 Selection of the appropriate method

The choice of an appropriate method of analysis for any particular case must be based on a balanced judgment of the advantages and inconveniences offered by each of the above methods.

The method of present value of costs has been selected in this study for two main reasons. Firstly, it allows for an appropriate identification of the costs without an undue investment in time to estimate, compute and refine all the input data, especially the benefits. Secondly, it is possible to compare the results in an ordinal manner to which one may associate limits of confidence.

It is important to note that with the selected model:

- The "do nothing" decision has not been considered part of the design process. It is assumed that this option should be rejected;

- Identical direct and indirect benefits are assumed to occur for each type of pavement. Of course, this is not necessarily true in reality. However, the assumption avoids the difficult problem of estimating benefits;

- It is assumed that a cost analysis will be made at a later date, that will determine precisely the real costs and financing methods of the optimal design.

6.3 Objectives of the selected method

The objectives of the selected method of economic analysis are:

a) To compare the costs of different pavement designs;

b) To examine the sensitivity of costs to variation in discount and interest rates; and

c) To identify the optimal design for a given situation.

6.3.1 Cost comparison

To compare the costs of different pavement designs, they must be based on the same dollar value and the same unit ($/yd^2 or $/m^2), for a given design period. It then becomes possible to identify the advantages and disadvantages of a given design in comparison to another, according to the set objectives.

6.3.2 Sensitivity analysis

The objective of a sensitivity analysis is to determine the influence on costs affected by changes in basic data. The analysis presented in this study will reflect primarely the influence of variations in the inflation and discount rate.

6.3.3 Selection of optimal design

The choice of optimal design is based on many factors, one of which is an economic analysis. The selected method of analysis will keep all useful information, while other material judged as non-pertinent or too expensive to obtain have simply been discarded.

6.4 Constants and variables

For the present value of costs method, the following variables require definition:

- Initial Construction Costs;
- Rehabilitation costs;
- Annual maintenance costs;
- Residual or salvage value; and
- To compare the total cost of each design one must also know the following:
- Inflation rate;
- Interest or discount rate;
- Period of the analysis.

6.4.1 Inflation rate

Inflation is the increase with time in the real value of available goods or services.

There are many ways to evaluate the rate of inflation but the most commonly used is the consumer Price Index (CPI). Since the future real costs of goods and/or services are influenced by increases in prices that are not related to the market requirements, it is imperative to take the inflation rate into account in the analysis.

6.4.2 Discount rate

The discount rate represents the social rate of return or the rate at which society discounts its future flow of earnings. This rate is quite different than interest rates. Government bond rates are often used as substitutes for the discount rate.

6.4.3 Period of analysis

For a comparison to be valid, costs must be analysed over a given time interval. For pavements this period of time is generally 20 to 30 years. The longer the period studied, the greater the likelihood of estimation error and the greater the uncertainly.

6.4.4 Initial construction costs

The initial construction costs are the sum of direct and indirect costs of construction. Direct costs include those for material, equipment and labour, while indirect costs include such items as frost protection blankets and drainage. Since in most cases the indirect costs are identical for all the design alternatives envisaged, they are excluded from this analysis.

6.4.5 Rehabilitation costs

Other than routine annual repairs, rehabilitation is any maintenance required to return the pavement to an acceptable service level. The major types of rehabilitation activities are:

- Planning;
- Reinforcement or strengthening;
- Resurfacing;
- Major Surface patching;
- Recycling.

For the purpose of this analysis, it is necessary to determine for each design to be compared:

- The type of rehabilitation required;
- The year, or years, in which the maintenance is to be done;
- The cost estimate.

6.4.6 Annual maintenance costs

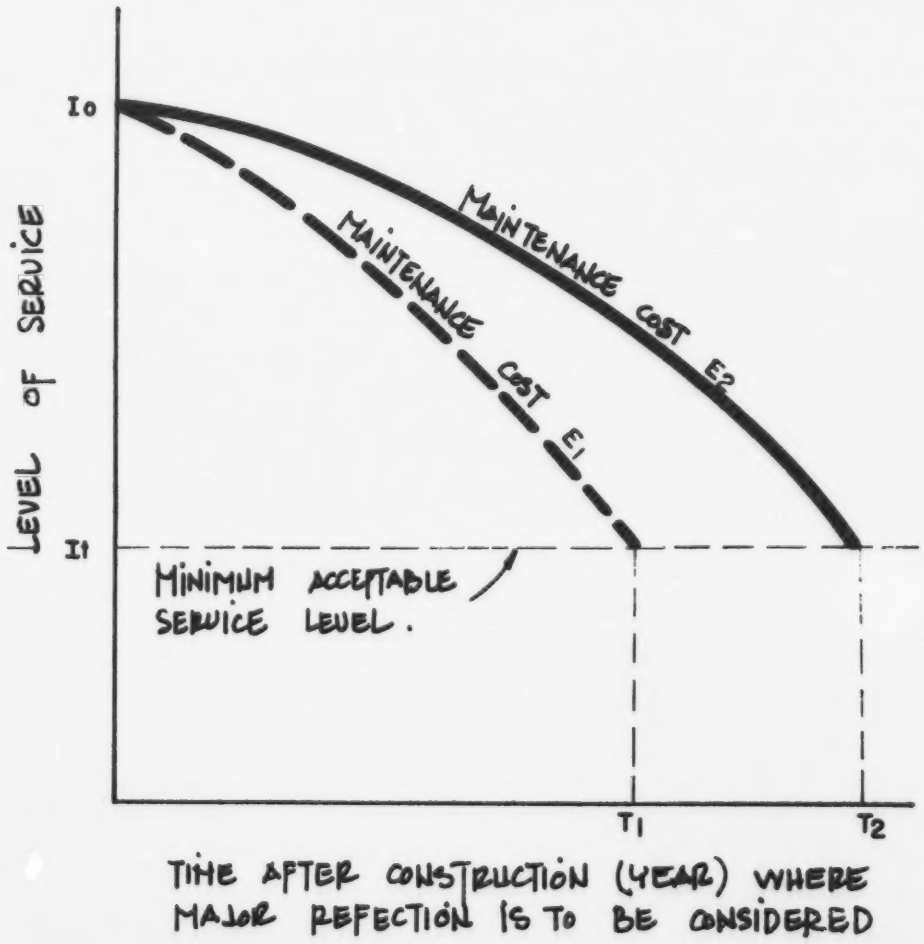
Routine maintenance consists of annual repairs to pavement defects that maintain a desired quality level and protect the pavement from accelerated deterioration. The type, the nature and the quality of annual maintenance repairs affect the rate of deterioration and also the life-span of the pavements. That is the performance of a pavement depends on the quality of its maintenance. Figure 6.1 illustrates the relationship between the cost of maintenance and its service level and life: with maintenance costs of E_1,the service level of the road follows curve,E_1,until it crosses the minimum acceptable level at time, T_1. At this point, it is necessary to rehabilitate the pavement to return the service level to its initial position at I_o. However, if annual maintenance costs were increased, the pavement would deteriorate along curve E_2 and rehabilitation would be delayed by T_2-T_1.

6.4.7 Residual value

The residual or salvage value is the value in dollars of the pavement at the end of the period of analysis.

FIGURE 6.1

INFLUENCE OF MAINTENANCE COST ON LEVEL OF SERVICE



When the analysis period corresponds to the useful life of the pavement, the salvage value represents the value of the recuperable materials. However, when the useful life of the pavement is greater than the analysis period, the residual value at the end of the analysis period equals both the value of recuperable materials and the remaining depreciated value of the pavement.

6.5 ANEC Program

A computerized program based on the present value of cost, has been developed to carry out precisely and quickly the economic analysis.

The programm, named ANEC, is illustrated in Figure 6.2 and was prepared:

a) To compute the total costs for each alternative design using different combinations of inflation and discount rates;

b) To present in graphic form:

- the cost variations of the design alternatives under consideration,
- the cost variations, given different levels of inflation.

The total cost of each design is computed by using the following formulae:

$$P_{vi} = C_i + \sum_{b}^{m} M_{il} \left(\frac{1+r}{1+a}\right) + \sum_{pi}^{h} A_{ip} \left(\frac{1+r}{1+a}\right)^p - R \left(\frac{1+r}{1+a}\right)^n$$

where:

P_{vi} = Present value of costs for design i

C_i = Initial construction cost of design i

M_{il} = Rehabilitation cost in year ℓ of design i

r = Inflation rate

a = Discount rate

A_{ip} = Annual maintenance costs in year "p" for design i

R = Residual value

n = Analysis period

The flow chart in figure 6.3 show the step followed in the computations. The programm has the capacity to compare a maximum of 10 designs associated with 5 inflation values and 5 discount rates. This range in inflation and discount rates permits the definition of minimum and maximum risks.

FIGURE 6.2

STRUCTURE OF ANEC PROGRAM

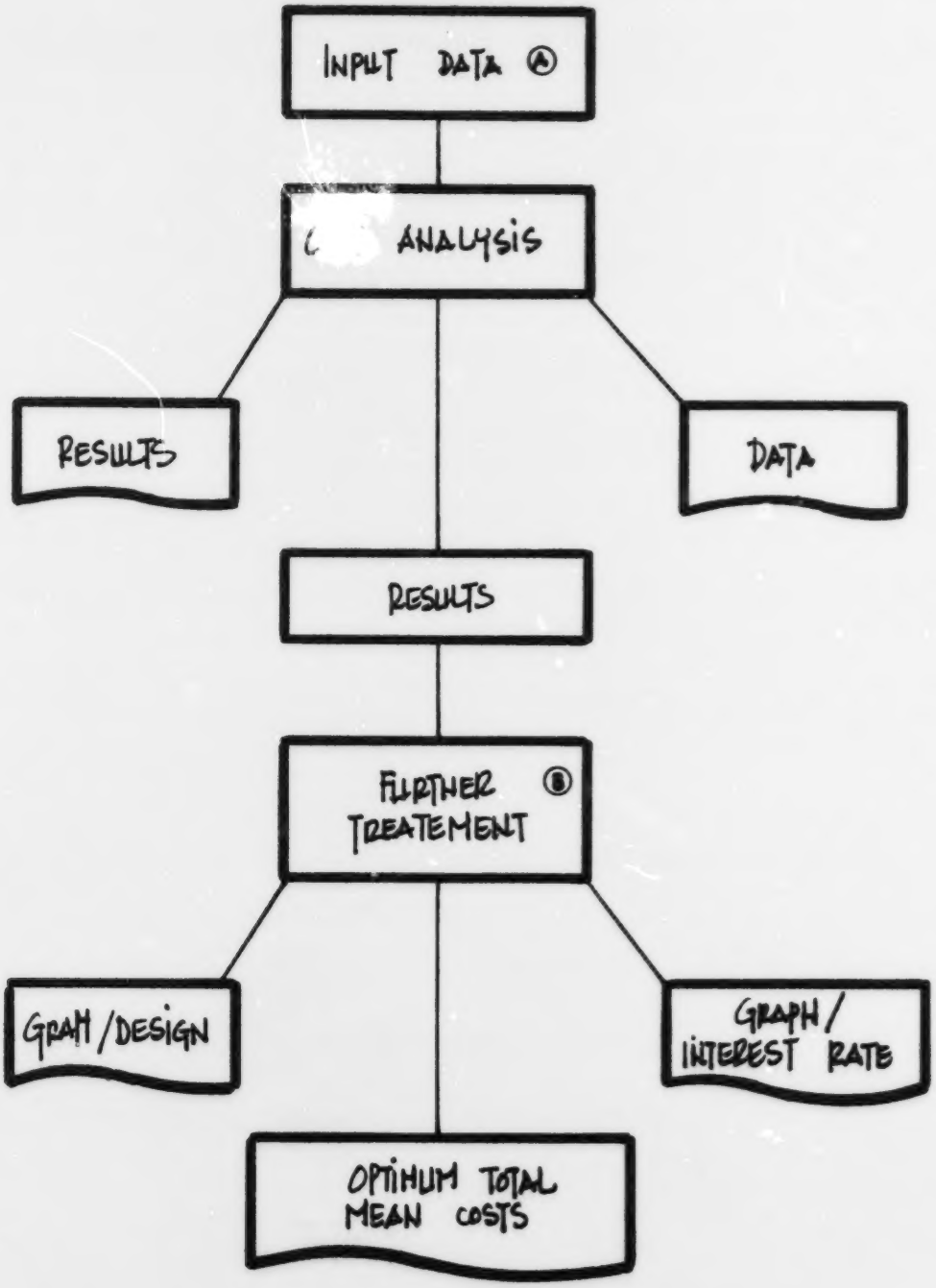
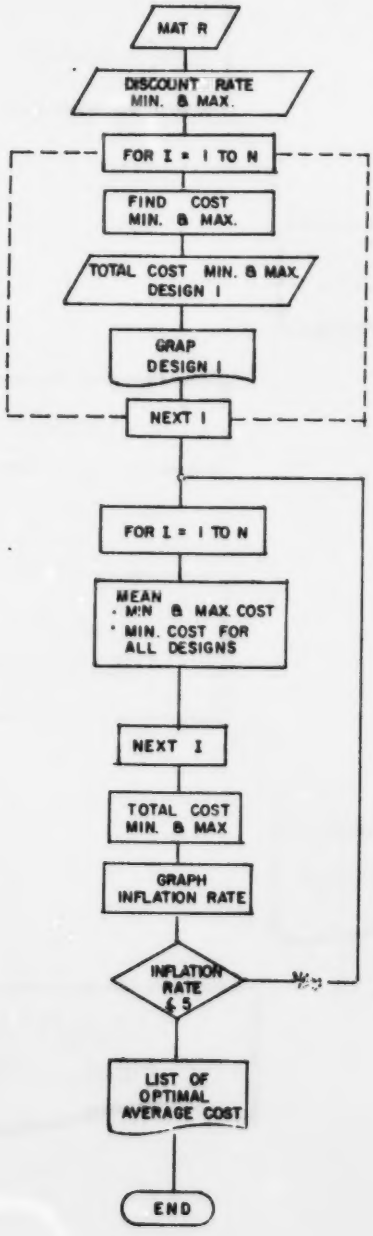


FIGURE 6.3

FLOW CHART SHOWING

THE STEPS FOLLOWED IN THE COMPUTATION



The results of the analysis are illustrated on charts. Figure 6.4 illustrates how, for a given design option, total design costs vary as a function of inflation and discount rates. It illustrates also the minimum and maximum risks associated with a given design.

Figure 6.5 shows the cost of each design and permits:

a) Comparison of the total costs of the different design alternatives as a function of the discount rate, while holding the inflation rate constant;

b) Determination of the minimum and maximum values for each design.

Since the choice of inflation and discount rates are somewhat arbitrary due to the difficulty inherent in forcasting, the ANEC programm incorporates an additional feature that can facilitate decision making. This feature is a value found by the Laplace Method and is defined as the authentic mean of all the costs of a design strategy:

$$\text{Max}_i \left[E\left(\Omega_{ij}\right)\right] = \text{Max}_i / \Sigma_j \frac{1}{n} \Omega_{ij} /$$

where:

n = possible number of costs for a given design; and

Ω_{ij} = total cost of a design at a discount rate of "j" and an inflation rate of "i".

With an average optimal cost, one assumes that the probability of occurence for each level of inflation is equal. The main advantage of this method is that all the data from the matrix are used and that it is based on probability rather than extreme values.

Illustrative example

It is desired to determine the variation of costs, with time, of 3 design alternatives for the pavement of a container terminal (Racine Terminal). The three alternatives under consideration are:

1. A pavement composed of paving blocks and bedding sand, resting on a 200 mm granular base course and a 780 mm foundation;

2. A flexible pavement composed of a 160 mm layer of bituminous mixture, 170 mm granular base and 780 mm foundation;

3. A rigid pavement made of paving blocks and bedding sand, on a 360 mm cement-stabilized granular base.

The details of each design are illustrated in figure 6.6. The cost results for each solution are as follows (the analysis period being set at 40 years):

FIGURE 6.4

INFLUENCE OF DISCOUNT RATE AND INFLATION RATE ON DIFFERENT DESIGN STRATEGIES.

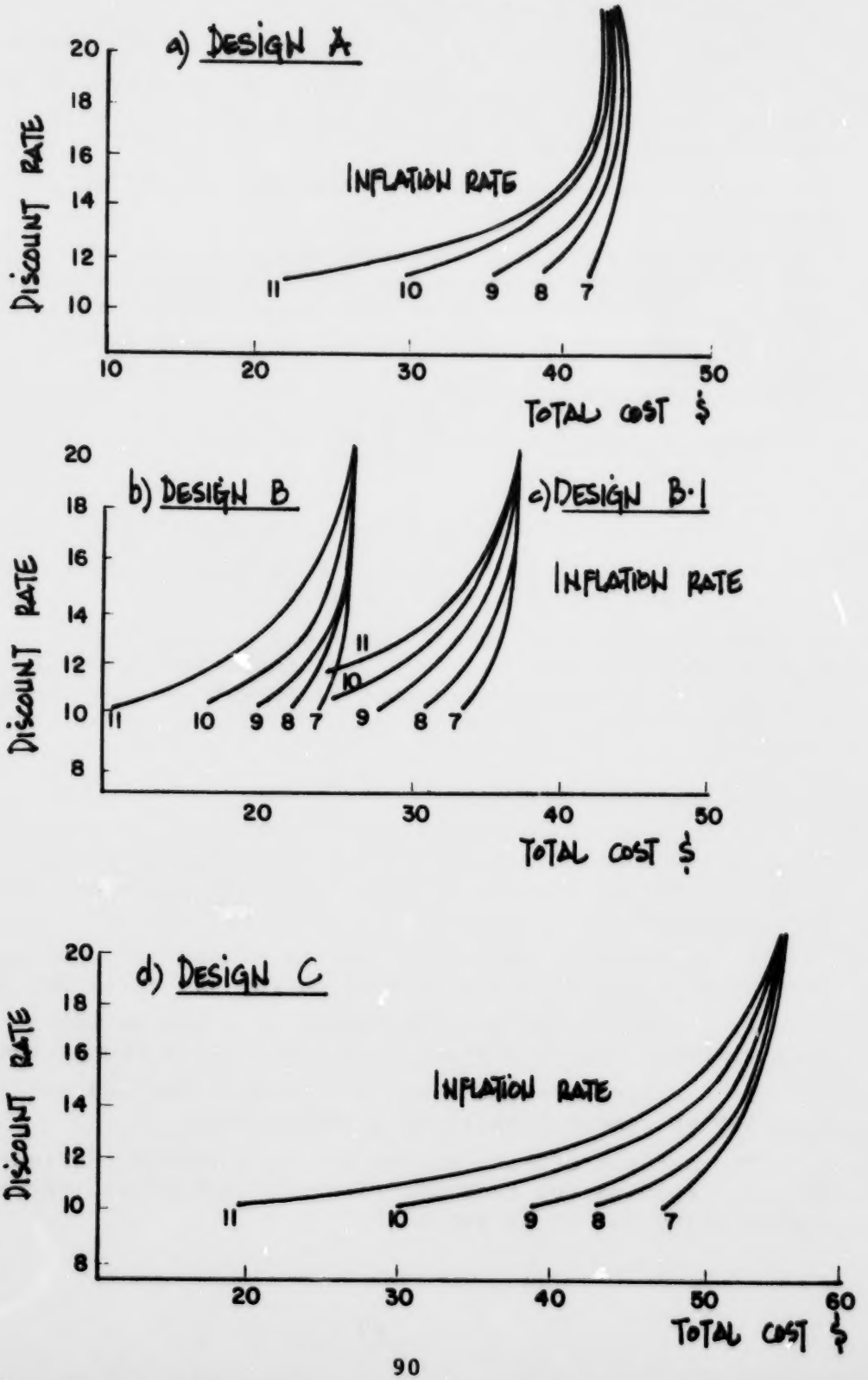
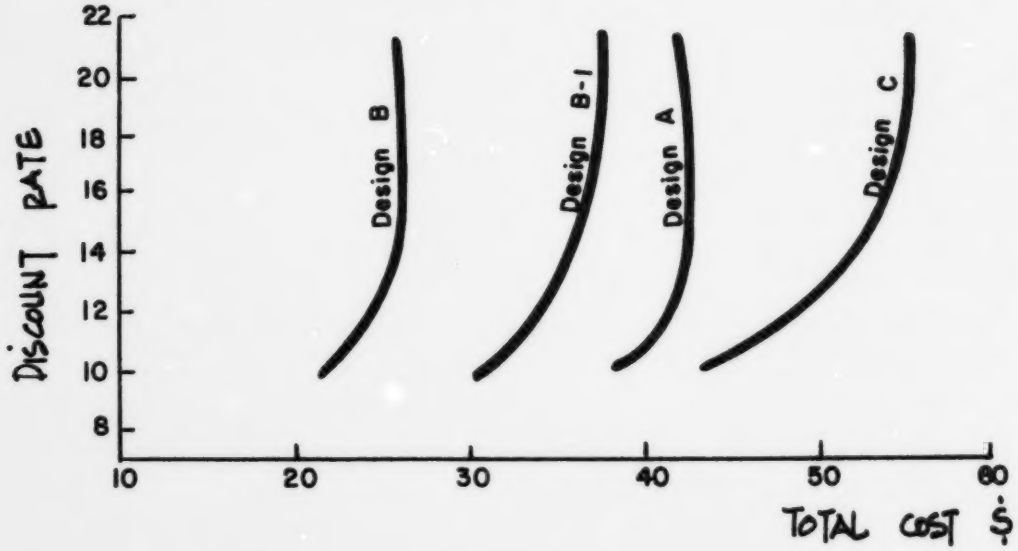
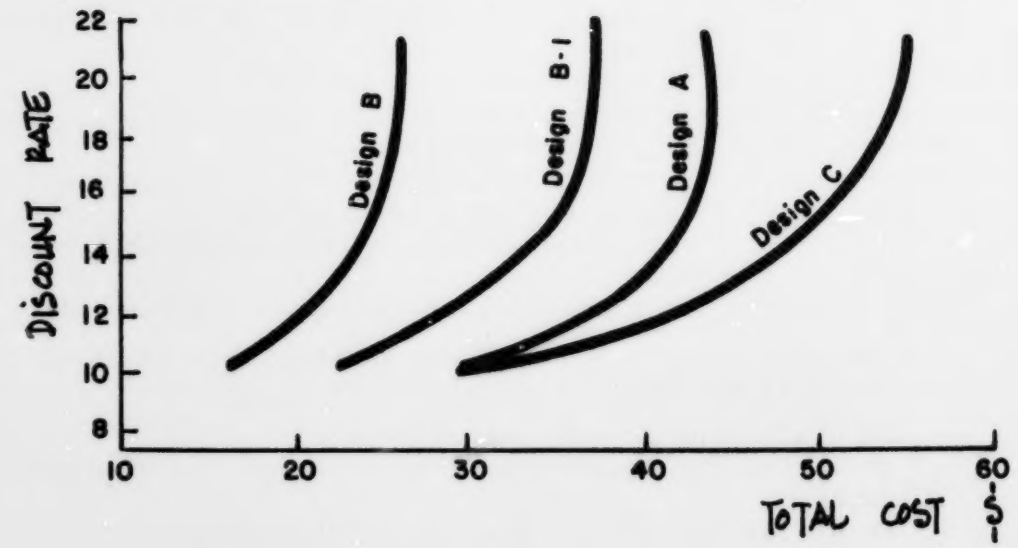


FIGURE 6.5
VARIATION OF TOTAL COSTS OF DIFFERENT DESIGN ALTERNATIVES.

a) INFLATION RATE 8%



b) INFLATION RATE 10%



Construction costs	$1/m^2
Alternative A	41,43
Alternative B	25,29
Alternative C	55,95
Maintenance costs	
Alternative A	0,03
Alternative B	0,14
Alternative C	nil
Rehabilitation costs	
Alternative A	13,45
Alternative B	1,61
Alternative C	nil
Salvage values	$1/m^2
Alternative A	25,29
Alternative B	15,60
Alternative C	25,29

With the following assumed inflation and discount rates:

Inflation: 7, 8, 9, 10 and 11%;
Discount: 10, 12, 15, 18 and 21%;

the output of the ANEC programm is given in the following tables and figures:

Table 6.1: summary of data on alternative designs;
Table 6.2: summary of total costs for each alternative design alternatives;
Table 6.3: strategy that yield the optimal average cost;

Figure 6.4a): variation of total costs as a function of interest and discount rates for alternative A;
Figure 6.4b): as above, for alternative B;
Figure 6.4c): as above, for alternative B-1;
Figure 6.4d): as above, for alternative C;
Figure 6.5a): variation of total costs of different alternatives as a function of the discount rate, the inflation rate being held constant at 8%;
Figure 6.5b): as above, for an inflation rate of 10%.

The results show that among the various alternatives, alternative B is the most rational since:

- it yields the lowest initial cost,
- it varies the least with the inflation and discount rates,
- it yields the lowest present value of costs.

Moreover, the impact of an increase in the price of bitumen which would double the cost of asphaltic mixtures, and increase the residual value of the mix from 1.08$ to 5.38 $/m^2, has been considered in alternative B-1. The results show that alternative B (or B-1) still remains the optimal solution. Also, as shown in Table 6.3, design B yields the optimum average cost for interest rate ranging from 7 to 11%.

Of course, in certain instances, another design strategy could be chosen but for reasons other than economic.

TABLE G1

EXAMPLE OF COSTS FOR THE DIFFERENT DESIGN SUGGESTED

DESIGN	CONSTRUCTION COST	MAINTENANCE COST	RESIDUAL VALUE	LENGTH OF PERIOD OF ANALYSIS	REFECTION	
					COST	TIME (YEARS)
A	41.43	.03	25.29	40.00	13.45	20
B	25.29	.14	15.60	40.00	1.61 1.61 1.61	10 20 30
B-1	36.00	.14	20.00	40.00	1.61 1.61 1.61	10 20 30
C	55.95	0.00	25.29	40.00	0.00	0

RATES :

INFLATION : 7.00 %, 8.00 %, 9.00 %, 10.00 %, 11.00 %
ACTUALISATION: 10.00 %, 12.00 %, 15.00 %, 18.00 %, 21.00 %

TABLE 6.2

EXAMPLE OF TOTAL COST FOR THE DIFFERENT DESIGN SUGGESTED

DESIGN	INFLATION RATE	ACTUALISATION RATE				
		10.00	12.00	15.00	18.00	21.00
A	7.00	41.11	43.02	43.41	43.00	42.55
	8.00	38.93	42.29	43.42	43.17	42.70
	9.00	35.40	40.98	43.29	43.30	42.86
	10.00	29.91	38.78	42.90	43.39	43.03
	11.00	21.55	35.28	42.14	43.38	43.18
B	7.00	24.44	26.09	26.76	26.73	26.54
	8.00	22.66	25.33	26.59	26.73	26.58
	9.00	19.99	24.16	26.29	26.69	26.61
	10.00	16.03	22.38	25.80	26.59	26.63
	11.00	10.21	19.72	25.02	26.41	26.62
B1	7.00	33.70	36.09	37.23	37.35	37.22
	8.00	31.26	35.01	36.94	37.31	37.24
	9.00	27.65	33.38	36.48	37.22	37.26
	10.00	22.34	30.95	35.76	37.04	37.24
	11.00	14.60	27.36	34.66	36.73	37.19
C	7.00	47.58	51.88	54.54	55.45	55.77
	8.00	43.81	50.05	53.90	55.22	55.68
	9.00	38.40	47.41	52.98	54.89	55.56
	10.00	30.66	43.65	51.68	54.42	55.39
	11.00	19.63	38.28	49.81	53.76	55.15

TABLE 6.3

ACTION GIVING A OPTIMUM MEAN COST

INFLATION RATE %	OPTIMUM ACTION
7	B
8	B
9	B
10	B
11	B

ANNEX SECTION 6

COMPUTER PROGRAM LISTING FOR COSTS ANALYSIS (ANEC)-1

```
0010 USE R(50,5),D$(17),J(5),I(5),J1
0020 : SAVE15,'COUTS - 1'
0030 :|  #   |   ###.##   | ###.##  |  ###.##  | ###.##  |
0040 :|      |            |         |          |        #|
0050 DIM D(10,4),E(10,4),A(10,4)
0060 DATA 'A','B','C','D','E','F','G','H','I','J','.','S'
0061 DATA '1','2','3','4','5'
0070 MAT READ D$
0080 L=10
0081 P$=X'73'
0090 PRINT 'ENTRER LE NOMBRE DE DESIGN'
0100 INPUT N
0101 IF N>10!N<0 GOTO 0090
0110 PRINT 'ENTRER L''INFLATION (5 VALEURS)'
0120 INPUT I(1),I(2),I(3),I(4),I(5)
0130 PRINT 'ENTRER LE TAUX D''ACTUALISATION (5 VALEURS)'
0140 INPUT J(1),J(2),J(3),J(4),J(5)
0150 FOR J=1 TO 5
0160 I(J)=I(J)/100
0170 J(J)=J(J)/100
0180 NEXT J
0190 MAT D(N,4)=(0)
0200 FOR B=1 TO N
0210 PRINT
0220 PRINT '-------------------------------------------------'
0230 PRINT '              DESIGN ';B;
0240 PRINT
0250 PRINT 'ENTRER:    - COUT DE CONSTRUCTION'
0260 PRINT '           - COUT D''ENTRETIEN ANNUEL'
0270 PRINT '           - VALEUR RESIDUELLE'
0280 PRINT '           - PERIODE D''ANALYSE'
0290 INPUT D(B,1),D(B,2),D(B,3),D(B,4)
0300 FOR J=1 TO 4
0310 IF J=1 GOTO 0330
0320 IF E(B,J-1)=0&A(B,J-1)=0 GOTO 0370
0330 PRINT 'ENTRER   - ENTRETIEN MAJEUR';J
0340 PRINT '         - ANNEE DE L''ENTRETIEN'
0350 INPUT E(B,J),A(B,J)
0360 NEXT J
0370 NEXT B
0380 GOSUB 1360
0390 FOR I=1 TO N
0400 FOR J=1 TO 4
0410 IF J=1 GOTO 0440
0420 IF E(I,J)=0&A(I,J)=0 GOTO 0520
0430 IF J>1 GOTO 0470
0440 PRINT FLP,TAB(10);
0450 PRINT USING FLP,0030,D$(I),D(I,1),D(I,2),D(I,3),D(I,4);
0460 GOTO 0490
0470 PRINT FLP,TAB(10);
0480 PRINT USING FLP,0040,' ';
0490 :###.##  |  ##   |
0500 PRINT USING FLP,0490,E(I,J),A(I,J)
```

```
0510 X9=X9+1
0520 NEXT J
0530 GOSUB 1550
0540 NEXT I
0550 PRINT FLP,
0560 PRINT FLP,TAB(10);'TAUX CONSIDERES : ';
0570 PRINT USING FLP,0600,'INFLATION',I(1)*100,P$,I(2)*100,P$;
0571 PRINT USING FLP,0610,I(3)*100,P$,I(4)*100,P$,I(5)*100,P$
0580 PRINT FLP,TAB(28);
0590 PRINT USING FLP,0600,'ACTUALISATION',J(1)*100,P$,J(2)*100,P$;
0591 PRINT USING FLP,0610,J(3)*100,P$,J(4)*100,P$,J(5)*100,P$
0600 :################: ##.## #, ##.## #,
0610 : ##.## #, ##.## #, ##.## #
0620 X9=X9+2
0670 GOSUB 1320
0680 MAT R(50,5)=(0)
0690 J1=1
0700 FOR J=1 TO N
0710 FOR B1=1 TO 5
0720 I=I(B1)
0730 GOSUB 0810
0740 NEXT B1
0750 NEXT J
0760 GOSUB 1040
0770 GOSUB 1590
0780 GOSUB 1320
0790 CHAIN 'E80',16
0800 END
0810 PRINT '-------------------------------------------------'
0820 PRINT '                INFLATION ';I*100
0830 PRINT '-------------------------------------------------'
0840 FOR K=1 TO 5
0850 A=J(K)
0860 R(J1,K)=D(J,1)
0870 FOR L1=1 TO 4
0880 IF E(J,L1)=0 GOTO 0900
0890 R(J1,K)=R(J1,K)+E(J,L1)*((1+I)/(1+A))↑A(J,L1)
0900 NEXT L1
0910 FOR O=1 TO D(J,4)
0920 R(J1,K)=R(J1,K)+D(J,2)*((1+I)/(1+A)↑O)
0930 NEXT O
0940 R(J1,K)=R(J1,K)-D(J,3)*((1+I)/(1+A))↑D(J,4)
0950 :DESIGN:##  INTERET:##.##  COUT TOTAL: #######.##  J1: ##
0960 PRINT USING 0950,J,J(K)*100,R(J1,K),J1
0970 IF R(J,K)≤G GOTO 0990
0980 G=R(J,K)
0990 IF R(J,K)≥P GOTO 1010
1000 P=R(J,K)
```

```
1010 NEXT K
1020 J1=J1+1
1030 RETURN
1040 X$=X'73'
1060 PRINT FLP,TAB(15);'VARIATIONS DES COUTS TOTAUX';
1080 PRINT FLP,' EN FONCTION DES TAUX D''ACTUALISATION'
1120 X9=2
1130 GOSUB 1860
1140 PRINT FLP,TAB(L);'I          I TAUX '      I';TAB(45);'TAUX';
1150 PRINT FLP,' D''INTERET';TAB(81);'I'
1160 PRINT FLP,TAB(L);'I DESIGN I D''          +';
1170 GOSUB 1260
1180 : #####.## I
1190 PRINT FLP,TAB(L);'I          I INFLATION I';
1200 FOR I5=1 TO 5
1210 PRINT USING FLP,1180,J(I5)*100;
1220 NEXT I5
1230 PRINT FLP,TAB(132);''
1240 :I    #      I    ##.##    I
1250 RETURN
1260 FOR I5=1 TO 5
1270 PRINT FLP,'----------+';
1280 NEXT I5
1290 PRINT FLP,TAB(132);''
1300 X9=X9+1
1310 RETURN
1320 FOR X1=1 TO 66-X9
1330 PRINT FLP,
1340 NEXT X1
1350 RETURN
1360 PRINT FLP,
1390 PRINT FLP,TAB(30);'TABLEAU DES DONNEES'
1400 PRINT FLP,TAB(30);'--------------------'
1410 PRINT FLP,
1420 PRINT FLP,
1430 PRINT FLP,
1440 PRINT FLP,TAB(10);'+-------+-----------+----------+';
1450 PRINT FLP,'----------+---------+-----------------+'
1460 PRINT FLP,TAB(10);'I       ICOUT          IENTRETIENI';
1470 PRINT FLP,'VALEUR    IPERIODE  IENTRETIEN MAJEURI'
1480 PRINT FLP,TAB(10);'IDESIGNICONSTRUCTIONIANNUEL    I';
1490 PRINT FLP,'RESIDUELLEID''ANALYSE+--------+--------+'
1500 PRINT FLP,TAB(10);'I       I             I          I';
```

```
1510 PRINT FLP,'          |          |  COUT  | ANNEE |'
1520 X9=10
1530 GOSUB 1550
1540 RETURN
1550 PRINT FLP,TAB(10);'+------+----------+---------+';
1560 PRINT FLP,'----------+---------+--------+--------+'
1570 X9=X9+1
1580 RETURN
1590 FOR I=1 TO J1-1
1600 :|   #    |   ##.##   |
1610 : ####.## |
1620 IF I=1|I=6 GOTO 1700
1630 IF I=11|I=16 GOTO 1700
1640 IF I=21|I=26 GOTO 1700
1650 IF I=31|I=36 GOTO 1700
1660 IF I=41|I=46 GOTO 1700
1670 PRINT FLP,TAB(L);
1680 PRINT USING FLP,1600,' ',I(D8)*100;
1690 GOTO 1760
1700 PRINT FLP,TAB(L);'+--------+----------+';
1710 GOSUB 1260
1720 D8=1
1730 D9=((I-1)/5)+1
1740 PRINT FLP,TAB(L);
1750 PRINT USING FLP,1600,D$(D9),I(D8)*100;
1760 FOR I5=1 TO 5
1770 PRINT USING FLP,1610,R(I,I5);
1780 NEXT I5
1790 PRINT FLP,TAB(132);''
1800 X9=X9+1
1810 D8=D8+1
1820 NEXT I
1830 PRINT FLP,TAB(L);'+--------+----------+';
1840 GOSUB 1260
1850 RETURN
1860 PRINT FLP,TAB(L);'+--------+----------+';
1870 FOR I5=1 TO 4
1880 PRINT FLP,'----------';
1890 NEXT I5
1900 PRINT FLP,'----------+'
1910 X9=X9+1
1920 RETURN
```

```
0010 :|   #    |    ###.##    |   ###.##  |  ###.##    |  ###.##   |
0020 :|        |              |           |            |           #|
0030 DIM D(10,4),E(10,4),A(10,4),R(10,5),D$(12),J(5),I(5)
0040 DIM W(21,101)
0050 DATA 'A','B','C','D','E','F','G','H','I','J','.','S'
0060 MAT READ D$
0070 MAT W(21,101)=(11)
0071 L=10
0072 :
0073 :
0074 :
0080 REM ************************************************************
0090 REM *                  ENTREE DES DONNEES                      *
0100 REM *                                                          *
0110 REM *   VARIABLE           IDENTIFICATION                      *
0120 REM *    MAT D        COL. 1: COUT CONSTRUCTION                *
0130 REM *                      2: COUT D'ENTRETIEN ANNUEL          *
0140 REM *                      3: VALEUR RESIDUELLE                *
0150 REM *                      4: PERIODE D'ANALYSE                *
0160 REM *   MAT E          COUT D'ENTRETIEN MAJEUR (MAX 4)         *
0170 REM *   MAT A          ANNEE DE L'ENTRETIEN MAJEUR (MAX 4)     *
0180 REM *   MAT I          %/% DE L'INFLATION (MAX 5)              *
0190 REM *   MAT J          %/% DU TAUX D'ACTUALISATION (MAX 5)     *
0200 REM *   N              NOMBRE DE DESIGN (MAX 10)               *
0210 REM ************************************************************
0220 PRINT 'ENTRER LE NOMBRE DE DESIGN'
0230 INPUT N
0240 PRINT 'ENTRER L''INFLATION (5 VALEURS)'
0250 INPUT I(1),I(2),I(3),I(4),I(5)
0260 PRINT 'ENTRER LE TAUX D''ACTUALISATION (5 VALEURS)'
0270 INPUT J(1),J(2),J(3),J(4),J(5)
0280 FOR J=1 TO 5
0290 I(J)=I(J)/100
0300 J(J)=J(J)/100
0310 NEXT J
0320 MAT D(N,4)=(0)
0330 FOR B=1 TO N
0340 PRINT
0350 PRINT '-------------------------------------------------'
0360 PRINT '                DESIGN ';B;
0370 PRINT
0380 PRINT 'ENTRER:    - COUT DE CONSTRUCTION'
0390 PRINT '           - COUT D''ENTRETIEN ANNUEL'
0400 PRINT '           - VALEUR RESIDUELLE'
0410 PRINT '           - PERIODE D''ANALYSE'
0420 INPUT D(B,1),D(B,2),D(B,3),D(B,4)
0430 FOR J=1 TO 4
0440 IF J=1 GOTO 0460
0450 IF E(B,J-1)=0&A(B,J-1)=0 GOTO 0500
0460 PRINT 'ENTRER   - ENTRETIEN MAJEUR';J
0470 PRINT '         - ANNEE DE L''ENTRETIEN'
0480 INPUT E(B,J),A(B,J)
0490 NEXT J
0500 NEXT B
```

```
0502 :
0503 :
0504 REM *******  P* TABLEAU DES DONNEES ***********
0510 GOSUB 2390
0520 FOR I=1 TO N
0530 FOR J=1 TO 4
0540 IF J=1 GOTO 0570
0550 IF E(I,J)=0&A(I,J)=0 GOTO 0640
0560 IF J>1 GOTO 0600
0570 PRINT FLP,TAB(10);
0580 PRINT USING FLP,0010,I,D(I,1),D(I,2),D(I,3),D(I,4);
0590 GOTO 0620
0600 PRINT FLP,TAB(10);
0610 PRINT USING FLP,0020,' ';
0620 : ###.##  |   ##   |
0630 PRINT USING FLP,0620,E(I,J),A(I,J)
0631 X9=X9+1
0640 NEXT J
0650 GOSUB 2590
0660 NEXT I
0670 PRINT FLP,
0680 PRINT FLP,TAB(30);'     TAUX CONSIDERES'
0690 PRINT FLP,TAB(30);'-------------------------'
0700 PRINT FLP,TAB(30);'INFLATION   ACTUALISATION'
0710 PRINT FLP,TAB(30);'---------   -------------'
0720 :    ##            ##
0721 X9=X9+5
0730 FOR I=1 TO 5
0740 PRINT FLP,TAB(30);
0750 PRINT USING FLP,0720,I(I)*100,J(I)*100
0751 X9=X9+1
0760 NEXT I
0761 GOSUB 1600
0770 REM
0771 REM
0780 REM
0781 REM
0790 REM ************************************************************
0800 REM *         COUT TOTAL AVEC INFLATION VARIABLE               *
0810 REM *                                                          *
0820 REM *   VARIABLE            IDENTIFICATION                     *
0830 REM *    MAT R           RESULTATS F(INTERETS VARIABLES)       *
0840 REM ************************************************************
0841 REM
0842 REM
0850 MAT R(N,5)=(0)
0860 FOR B1=1 TO 5
0870 I=I(B1)
0880 GOSUB 1020
0890 GOSUB 1220
0900 GOSUB 1600
0910 GOSUB 1640
0920 GOSUB 2200
0930 GOSUB 2020
0940 GOSUB 2100
0950 GOSUB 1930
0960 GOSUB 2100
0970 GOSUB 2020
0980 GOSUB 1600
0990 NEXT B1
0991 :
0992 :
1000 END
```

```
1001 REM L'INSTRUCTION PRECEDENTE INDIQUE LA FIN DU PROGRAMME
1002 .
1003 :
1004 :
1010 REM **********  SOUS-ROUTINE CALCUL DU COUT TOTAL  **********
1020 FOR J=1 TO N
1030 PRINT '-------------------------------------------------'
1040 PRINT '                INFLATION ';I*100
1041 PRINT '-------------------------------------------------'
1050 FOR K=1 TO 5
1060 A=J(K)
1070 R(J,K)=D(J,1)
1080 FOR L1=1 TO 4
1090 IF E(J,L1)=0 GOTO 1110
1100 R(J,K)=R(J,K)+E(J,L1)*((1+I)/(1+A))↑A(J,L1)
1110 NEXT L1
1120 FOR O=1 TO D(J,4)
1130 R(J,K)=R(J,K)+D(J,2)*((1+I)/(1+A)↑O)
1140 NEXT O
1150 R(J,K)=R(J,K)-D(J,3)*((1+I)/(1+A))↑D(J,4)
1160 :DESIGN: #  INTERET:##.##  COUT TOTAL: ######.##
1170 PRINT USING 1160,J,J(K)*100,R(J,K)
1171 IF R(J,K)≤G GOTO 1175
1172 G=R(J,K)
1175 IF R(J,K)≥P GOTO 1180
1176 P=R(J,K)
1180 NEXT K
1190 NEXT J
1200 RETURN
1201 :
1202 :
1203 :
1210 :   **********   P* TABLEAU DES RESULTATS      **********
1220 X$=X'73'
1230 PRINT FLP,
1231 PRINT FLP,
1232 PRINT FLP,
1240 PRINT FLP,TAB(20);'TABLEAU INDIQUANT LA VARIATION DES';
1250 PRINT FLP,' COUTS TOTAUX '
1260 PRINT FLP,TAB(20);'EN FONCTION DE DIVERS TAUX D''ACTUALI';
1270 PRINT FLP,'SATION'
1280 PRINT FLP,
1290 PRINT FLP,TAB(30);'INFLATION EGALE A';I*100;X$
1300 PRINT FLP,
1310 PRINT FLP,
1320 X9=9
1330 GOSUB 1530
1340 PRINT FLP,TAB(L);'I   DESIGN   I';
1350 : ####.## I
1360 FOR I5=1 TO 5
1370 PRINT USING FLP,1350,J(I5)*100;
1380 NEXT I5
1390 PRINT FLP,TAB(132);''
1400 GOSUB 1530
1410 :I     #      I
1420 FOR J=1 TO N
1430 PRINT FLP,TAB(L);
1440 PRINT USING FLP,1410,D$(J);
1450 FOR K=1 TO 5
1460 PRINT USING FLP,1350,R(J,K);
1470 NEXT K
1480 PRINT FLP,TAB(132);''
1490 X9=X9+1
1500 NEXT J
```

```
1510 GOSUB 1530
1520 RETURN
1521 :
1522 :
1523 :    ******* P* LIGNE ----------+ **********
1530 PRINT FLP,TAB(L);'+------------+';
1540 FOR I5=1 TO 5
1550 PRINT FLP,'----------+';
1560 NEXT I5
1570 PRINT FLP,TAB(132);''
1580 X9=X9+1
1590 RETURN
1591 :
1592 :
1599 :   ************ SOUS-ROUTINE SAUT DE PAGE ***************
1600 FOR X1=1 TO 66-X9
1610 PRINT FLP,
1620 NEXT X1
1630 RETURN
1631 :
1632 :
1633 :
1640 :   ************ SOUS-ROUTINE P* GRAPHIQUE **************
1650 MAT W=(11)
1651 PRINT 'MINIMUM: ';J(1)*100,'MAXIMUM: ';J(5)*100
1660 PRINT
1670 PRINT 'ENTRER: - INTERET MINIMUM  ET MAXIMUM'
1680 PRINT '        - NBRE DE POSITIONS (MAX 20)'
1690 INPUT I1,I2,G1
1700 PRINT
1701 PRINT 'MINIMUM: ';P,'MAXIMUM: ';G
1710 PRINT 'ENTRER: - COUT TOTAL MINIMUM ET MAXIMUM'
1720 PRINT '        - NBRE DE POSITIONS (50,60,70,80,90 OU 100)'
1730 INPUT C1,C2,G2
1740 REM CALCUL DE LA VALEUR DE CHAQUE INTERVALLE (G3, G4)
1750 G3=(I2-I1)/G1
1760 G4=(C2-C1)/G2
1770 M=6
1780 FOR G=1 TO 5
1790 M=M-1
1800 FOR H=1 TO N
1810 Y1=INT(((R(H,G)-C1)/G4)+.05)+1
1820 X1=INT(((J(M)*100-I1)/G3)+.05)+1
1830 IF W(X1,Y1)=11 GOTO 1860
1840 W(X1,Y1)=12
1850 GOTO 1870
1860 W(X1,Y1)=H
1870 : COUT:###.### INTERET:##.#    X1:##    Y1:##
1880 PRINT USING 1870,R(H,G),J(M)*100,X1,Y1
1890 NEXT H
1900 NEXT G
1920 RETURN
1921 :
1922 :
1923 :   **********  P* UNE LIGNE DU GRAPHIQUE **********
1930 FOR I=1 TO G1+1
1940 PRINT FLP,TAB(L-1);'+';
1950 FOR J=1 TO G2
1960 PRINT FLP,TAB(L+J);D$(W(I,J));
1970 NEXT J
1980 PRINT FLP,'+'
1990 NEXT I
2000 X9=X9+1
```

```
2010 RETURN
2011 :
2012 :
2013 :
2020 REM **** SOUS-ROUTINE P* VALEURS AUX 10 INTERVALLES *****
2030 :     #######.##
2031 : #######.##
2040 PRINT FLP,TAB(L);
2050 FOR G5=1 TO G2/10
2051 IF G5>1 GOTO 2062
2060 PRINT USING FLP,2030,(C1+G5*10*G4);
2061 GOTO 2070
2062 PRINT USING FLP,2031,(C1+G5*10*G4);
2070 NEXT G5
2080 PRINT FLP,TAB(132);' '
2081 X9=X9+1
2090 RETURN
2091 :
2092 :
2093 :
2100 REM ---- SOUS-ROUTINE P* +++++++++, -----
2110 PRINT FLP,TAB(L);'.+++++++++.+++++++++.+++++++++.+++++++++';
2120 PRINT FLP,'.+++++++++.';
2130 IF G2/10=5 GOTO 2170
2140 FOR G5=6 TO G2/10
2150 PRINT FLP,'+++++++++.';
2160 NEXT G5
2170 PRINT FLP,TAB(132);' '
2180 X9=X9+1
2190 RETURN
2191 :
2192 :
2193 :
2200 REM **** SOUS-ROUTINE ENTETE DU GRAPHIQUE ****
2210 X9=13
2220 PRINT FLP,
2230 PRINT FLP,
2240 PRINT FLP,TAB(25);'FIGURE  '
2250 PRINT FLP,TAB(25);'---------'
2260 PRINT FLP,
2270 PRINT FLP,
2280 PRINT FLP,TAB(L+20);'GRAPHIQUE COMPARATIF ILLUSTRANT'
2290 PRINT FLP,TAB(L+15);'LA VARIATION DES COUTS TOTAUX DE DI';
2291 PRINT FLP,'FFERENTS DESIGNS'
2300 PRINT FLP,TAB(L+15);'EN FONCTION DE DIVERS TAUX D''INFLA';
2310 PRINT FLP,'TION ET D''ACTUALISATION.'
2320 PRINT FLP,
2330 PRINT FLP,
2340 PRINT FLP,TAB(L+11);' ';
2350 : CHAQUE '+' = ####.## UNITES
2360 PRINT USING FLP,2350,G4
2370 PRINT FLP,
2380 RETURN
```

```
2381 :
2382 :
2383 :
2390 :   ****** P* ENTETE TABLEAU DES DONNEES *******
2400 FOR I=1 TO 5
2410 PRINT FLP,
2420 NEXT I
2430 PRINT FLP,TAB(30);'TABLEAU DES DONNEES'
2440 PRINT FLP,TAB(30);'--------------------'
2450 PRINT FLP,
2460 PRINT FLP,
2470 PRINT FLP,
2480 PRINT FLP,TAB(10);'+---------+--------------+-----------+';
2490 PRINT FLP,'------------+-----------+-------------------+'
2500 PRINT FLP,TAB(10);'|         | COUT          | ENTRETIEN |';
2510 PRINT FLP,' VALEUR     | PERIODE   | ENTRETIEN MAJEUR |'
2520 PRINT FLP,TAB(10);'| DESIGN | CONSTRUCTION | ANNUEL    |';
2530 PRINT FLP,' RESIDUELLE | D''ANALYSE +----------+--------+'
2540 PRINT FLP,TAB(10);'|         |               |           |';
2550 PRINT FLP,'             |           |   COUT  |  ANNEE |'
2560 X9=14
2570 GOSUB 2590
2580 RETURN
2581 :
2582 :
2583 :
2590 :   ***** P* LIGNE ---------... DU TABLEAU DES DONNEES *****
2600 PRINT FLP,TAB(10);'+---------+--------------+-----------+';
2610 PRINT FLP,'------------+-----------+----------+--------+'
2620 X9=X9+1
2630 RETURN
```

```
0010 USE R(50,5),D$(17),J(5),I(5),J1
0020 :SAVE16,'COUTS - 2'
0030 DIM B(5),F(5),E(10),D(10),W(21,101),M(5)
0040 DATA 0,5,10,15,20,25,30,35,40,45
0050 MAT READ E
0060 DATA -4,-3,-2,-1,0
0070 MAT READ F
0080 MAT W(21,101)=(11)
0081 MAT M=(9999999999)
0090 L=15
0100 P$=X'73'
0110 REM
0120 REM ***************************************************************
0130 REM *         COUT TOTAL AVEC INFLATION VARIABLE                  *
0140 REM *                                                             *
0150 REM *    VARIABLE              IDENTIFICATION                     *
0160 REM *     MAT R             RESULTATS F(INTERETS VARIABLES)       *
0170 REM ***************************************************************
0180 REM
0190 PRINT
0200 PRINT 'MINIMUM: ';J(1)*100,'MAXIMUM: ';J(5)*100
0210 PRINT
0220 PRINT 'ENTRER: - INTERET MINIMUM  ET MAXIMUM'
0230 PRINT '        - NBRE DE POSITIONS (10,15 OU 20)'
0240 INPUT I1,I2,G1
0250 IF I1>J(1)*100!I2<J(5)*100 GOTO 0190
0260 IF G1=10!G1=20 GOTO 0290
0270 IF G1=15 GOTO 0290
0280 GOTO 0190
0290 N2=5
0300 N3=1
0310 MAT B=(5)
0320 FOR I=1 TO (J1-1)/5
0330 MAT F=F+B
0340 GOSUB 0960
0350 GOSUB 2010
0360 GOSUB 1740
0370 GOSUB 1880
0380 GOSUB 1530
0390 GOSUB 1880
0400 GOSUB 1740
0410 GOSUB 2450
0420 GOSUB 0890
0430 NEXT I
0440 MAT D=(1)
0450 N2=(J1-1)/5
0460 N3=2
0470 FOR I=1 TO 5
0480 MAT E=E+D
0490 GOSUB 0960
0500 GOSUB 2010
```

```
0510 GOSUB 1740
0520 GOSUB 1880
0530 GOSUB 1530
0540 GOSUB 1880
0550 GOSUB 1740
0560 GOSUB 2450
0570 GOSUB 0890
0580 NEXT I
0590 I1=I(1)*100
0600 I2=I(2)*100
0610 I3=I(3)*100
0620 I4=I(4)*100
0630 I5=I(5)*100
0640 :--------  ---------  ---------  --------  ---------
0650 :##.## #   ##.## #   ##.## #   ##.## #   ##.## #
0660 :#####.##   #####.##   #####.##   #####.##   #####.##
0670 :-----------------------------------------------------
0680 :                TAUX D'INFLATION
0681 PRINT FLP,
0682 PRINT FLP,
0690 PRINT FLP,TAB(30);'COUTS MOYENS OPTIMUM'
0700 PRINT FLP,TAB(30);'--------------------'
0710 PRINT FLP,
0711 PRINT FLP,
0720 PRINT FLP,TAB(15);
0730 PRINT FLP,TAB(15);'              TAUX D''INFLATION'
0740 PRINT FLP,TAB(15);'-----------------------------------------------------';
0750 PRINT FLP,'------'
0760 PRINT FLP,TAB(15);
0770 PRINT USING FLP,0650,I1,P$,I2,P$,I3,P$,I4,P$,I5,P$
0780 PRINT FLP,TAB(15);
0790 PRINT USING FLP,0640
0800 PRINT FLP,TAB(15);
0810 PRINT USING FLP,0660,M(1),M(2),M(3),M(4),M(5)
0820 :
0830 END
0840 REM L'INSTRUCTION PRECEDENTE INDIQUE LA FIN DU PROGRAMME
0850 :
0860 :
0870 :
0880 :    ************ SOUS-ROUTINE SAUT DE PAGE ***************
0890 FOR X1=1 TO 66-X9
0900 PRINT FLP,
0910 NEXT X1
0920 RETURN
0930 :
0940 :
0950 :
0960 :    *** SOUS-ROUTINE CALCUL DES VALEURS DU GRAPHIQUE ***
0970 P=999999999
0980 G=-999999999
```

```
0990 FOR K9=1 TO N2
1000 FOR B1=1 TO 5
1010 GOTO 1020,1070 ON N3
1020 IF R(F(K9),B1)≤G GOTO 1040
1030 G=R(F(K9),B1)
1040 IF R(F(K9),B1)≥P GOTO 1110
1050 P=R(F(K9),B1)
1060 GOTO 1110
1070 IF R(E(K9),B1)≤G GOTO 1090
1080 G=R(E(K9),B1)
1090 IF R(E(K9),B1)≥P GOTO 1110
1100 P=R(E(K9),B1)
1110 NEXT B1
1111 IF N3=1 GOTO 1150
1120 M=(R(E(K9),1)+R(E(K9),2)+R(E(K9),3)+R(E(K9),4)+R(E(K9),5))/5
1130 IF M≥M(I) GOTO 1150
1140 M(I)=M
1150 NEXT K9
1160 PRINT
1170 :        MINIMUM : ###.##          MAXIMUM : ###.##
1180 PRINT USING 1170,P,G
1190 PRINT
1200 PRINT 'ENTRER: - COUT TOTAL MINIMUM ET MAXIMUM'
1210 PRINT '        - NBRE DE POSITIONS (50,60,70,80,90 OU 100)'
1220 INPUT C1,C2,G2
1230 IF C1>P!C2<G GOTO 1160
1240 IF G2<50!G2>100 GOTO 1160
1250 MAT W=(11)
1260 P=999999999
1270 G=-999999999
1280 REM CALCUL DE LA VALEUR DE CHAQUE INTERVALLE (G3, G4)
1290 G3=(I2-I1)/G1
1300 G4=(C2-C1)/G2
1310 M=6
1320 FOR G=1 TO 5
1330 M=M-1
1340 FOR H=1 TO N2
1350 GOTO 1360,1380 ON N3
1360 Y1=INT(((R(F(H),G)-C1)/G4)+.05)+1
1370 GOTO 1390
1380 Y1=INT(((R(E(H),G)-C1)/G4)+.05)+1
1390 X1=INT(((J(M)*100-I1)/G3)+.05)+1
1400 IF W(X1,Y1)=11 GOTO 1430
1410 W(X1,Y1)=12
1420 GOTO 1470
1430 GOTO 1440,1460 ON N3
1440 W(X1,Y1)=H+12
1450 GOTO 1470
1460 W(X1,Y1)=H
1470 NEXT H
1480 NEXT G
1490 RETURN
1500 :
1510 :
```

```
1520 :   **********  P* UNE LIGNE DU GRAPHIQUE **********
1530 FOR K=1 TO G1+1
1540 P1=I1+G3*(K-1)
1550 IF K=1!K=6 GOTO 1590
1560 IF K=11!K=16 GOTO 1590
1570 I$='+'
1580 GOTO 1630
1590 PRINT FLP,TAB(L-8);' ';
1600 : ##.##
1610 PRINT USING FLP,1600,P1;
1620 I$='.'
1630 PRINT FLP,TAB(L-1),I$;
1640 FOR J=1 TO G2+1
1650 PRINT FLP,TAB(L-1+J);D$(W(K,J));
1660 NEXT J
1670 PRINT FLP,TAB(G2+1+L);I$
1680 X9=X9+1
1690 NEXT K
1700 RETURN
1710 :
1720 :
1730 :
1740 REM **** SOUS-ROUTINE P* VALEURS AUX 10 INTERVALLES *****
1750 :     #######.##
1760 : #######.##
1770 PRINT FLP,TAB(L-7);
1780 PRINT USING FLP,1760,C1;
1790 FOR G5=1 TO G2/10
1800 PRINT USING FLP,1760,(C1+G5*10*G4);
1810 NEXT G5
1820 PRINT FLP,TAB(132);' '
1830 X9=X9+1
1840 RETURN
1850 :
1860 :
1870 :
1880 REM ----- SOUS-ROUTINE P* +++++++++, ------
1890 PRINT FLP,TAB(L);',+++++++++,+++++++++,+++++++++,+++++++++';
1900 PRINT FLP,',+++++++++.';
1910 IF G2/10=5 GOTO 1950
1920 FOR G5=6 TO G2/10
1930 PRINT FLP,'+++++++++.';
1940 NEXT G5
1950 PRINT FLP,TAB(132);' '
1960 X9=X9+1
1970 RETURN
1980 :
1990 :
2000 :
2010 REM **** SOUS-ROUTINE ENTETE DU GRAPHIQUE ****
2020 X9=14
2030 PRINT FLP,
2040 PRINT FLP,
2050 PRINT FLP,TAB(40);'FIGURE  '
```

```
2060 PRINT FLP,TAB(40);'---------'
2070 PRINT FLP,
2080 PRINT FLP,
2090 PRINT FLP,TAB(L+20);' VARIATIONS DES COUTS TOTAUX'
2100 PRINT FLP,TAB(L+10);'EN FONCTION DES TAUX D''INFLA',
2110 PRINT FLP,'TION ET D''ACTUALISATION.'
2120 PRINT FLP,
2130 GOTO 2140,2320 ON N3
2140  DESIGN #
2150 PRINT FLP,TAB(L);
2160 PRINT USING FLP,2140,D$(I)
2170 PRINT FLP,
2180 X9=X9+4
2190 :COURBE   TAUX D'INFLATION
2200 :------   ----------------
2210 PRINT FLP,TAB(L);
2220 PRINT USING FLP,2190
2230 PRINT FLP,TAB(L);
2240 PRINT USING FLP,2200
2250 :   #          ##.## #
2260 FOR N=1 TO 5
2270 PRINT FLP,TAB(L);
2280 PRINT USING FLP,2250,N,I(N)*100,P$
2290 X9=X9+1
2300 NEXT N
2310 GOTO 2360
2320 :TAUX D'INFLATION : ##.## #
2330 PRINT FLP,TAB(L);
2340 PRINT USING FLP,2320,J(I)*100,P$
2350 X9=X9+1
2360 PRINT FLP,
2370 PRINT FLP,TAB(L+11);' ';
2380 : CHAQUE '+' =###.## UNITE(S)
2390 PRINT FLP,TAB(L+20);'COUTS TOTAUX'
2400 PRINT FLP,
2410 PRINT FLP,TAB(L+12);
2420 PRINT USING FLP,2380,G4
2430 PRINT FLP,
2440 RETURN
2450 PRINT FLP,
2460 PRINT FLP,TAB(L-6);'ACTUALISATION'
2470 PRINT FLP,
2480 PRINT FLP,TAB(L-7);
2490 PRINT USING FLP,2380,G3
2500 X9=X9+4
2510 RETURN
```

7. PERFORMANCE OF TEST SECTIONS DURING THE FIRST TWO YEARS, FROM OCTOBER 1981 TO DECEMBER 1983. (1)

7.1 Monitoring program

The monitoring program was designed to assess both the variations in the environmental conditions of pavements components layers and soils, and the degradation with time of the pavement surface.

- The environmental conditions of pavements layers were defined in terms of the variation of moisture and temperature. Three instrumented test sections A, C and D, were evaluated, principally during the winter and during the thawing periods. Readings were done on a regular basis, generally bi monthly, but, because of access problems on the busy site and the changing weather conditions, particularly during the winter of 1982 - 1983, the reading schedule had to be changed several times.
- The variation of bearing capacity was evaluated in terms of the variation of the surface deflections as measured with Dynaflect and also in terms of the dynamic moduli of layers calculated from the measured deflections.

Dynaflect deflections were measured five times after constructions;

- once at the end of construction (September 1981)
- twice in 1982; first during the summer (June), and again in the fall (November).
- twice in 1983; once in the summer (July) and once in the fall (October).

Measurements of deflection under actual gantry and container loading were also done with a Benkelman beam in the months of June and July, 1982. Deflections under wheel loads of up to 45 tons were measured at the site.

From these measurements, dynamic moduli for each pavement layers of different structures were calculated using a two layers elastic computer program and compared with the same measured at the end of construction.

Finally surface degradations were evaluated as four times since the end of construction: i.e. differentials settlements or heave cracking or spalling and scalling of concrete blocks and cracking of asphalt pavement.

Variation of profile of the pavements surface were evaluated by topographic survey while the deformation and rutting of the surface has been measured by a straight edge. The degree of cracking and spalling of concrete blocks have been evaluated visually by counting broken and/or scaled blocks.

7.2 Changes in thermal and moisture regime of the instrumented sections

Sections A, C, and D were instrumented (cf: section 5),

(1) with M.A. Laforte, p. eng.

section (A) is made of 100 mm block over crushed stone base, section (C) is made of a 160 mm asphaltic concrete layer over crushed stone base and section (D) is made of 80 mm block pavements over cement treated crushed stone base.

The performance of all sections have been evaluated by comparing the performance of different types of pavements structure (flexible, semi-rigid, and rigid) and by measuring the variation of the environment during winters and springs of 1981-1982 and of 1982-1983.

7.2.1 Climatic conditions of winter and spring following the construction

The winter that followed construction of the test pavements (1981-1982) was particularly severe, being one of the most severe Montreal ever had, in terms of Freezing Index (FI) , length of freezing time, mean winter temperature ans snowfall.

The freezing period lasted from December 4th 1981 to March 1st 1982 and the freezing index was 997 degree days (compared to mean maximum of 1000 degree days). Minimum temperature was -35 degrees Celsius (early in the morning of January 18th) and there was very few freezing-thaw periods from December to the end of February. There was a total of 35 air freeze-thaw cycles, principally during late November, early December, early March and early April.

7.2.2 Climatic conditions during winter and spring of the second year (1982-1983)

The second winter was, on the contrary, one of the mildest; with a freezing index of only 571 degree-day and frequent freeze-thaw periods during December, January and February. The longest continuous cold spell was about 10 days and the lowest temperature was 22 degrees Celsius, attained 3 times, once in December, once early in January and once in February. Spring came very early in the first days of March, followed by a cold, snowy and rainy period starting March 15 until late April. Snow was scarce but rain and freezing rain were abundant during the many thawing periods. There was all together 70 air freeze-thaw cycles with 28 from the period between December 15th and February 1st and 35 between February 15th and late April.

7.2.3 Thermal regime of the instrumented sections

Due to the highly variable weather conditions during the winter of 1982-1983 and the low number of readings, few measurements could be interpreted on a rational basis during that period. However, the results shown at the end of this section indicated the same trends as those of the first winter (1981-1982), the following comments are thus based on the results of the first winter which are shown on the figures hereafter.

The results will be interpreted in terms of frost depth, minimum surface pavement temperature and global temperature variation under the different pavements (asphalt pavement, block pavement on granular or cement treated base.

7.2.3.1 Frost depth

Frost penetration under the different types of pavement varied from 1,60 mm to 1,88 mm during the winter of 1981-1982 and from 1,38 m to 1,63 m during the winter of 1982-1983: the asphalt pavement had the least frost penetration during both seasons (160 and 135 cm, figure 7.1); the block pavement over concrete base had the deepest penetration (185 and 160 cm, figure 7.2); while the frost penetration under block pavement over granular materials was intermediate (170 and 150 cm, figure 7.3).

7.2.3.2 Temperature variation at surface and with depth

The differences in frost penetration can be explained by the ground temperature distribution: the black-top asphalt pavement had the warmest minimum temperature, -12 degrees Celsius at 50 mm. No delay response is noted up to 600 mm while the delay in temperature response increases at 800 mm and 1600 mm below the pavement surface (figure 7.4).

The lighter-coloured block pavement on cement stabilized base yields the lowest temperature both at the surface (-20 degrees Celsius at 50 mm) and in depth. It also yields a no delay response at greater depth (up to 1600 mm, figure 7.5).

The lighter-coloured block pavement on granular base yields a colder temperature between 100 mm and 600 mm and intermediate temperatures at greater depths. Delay response is similar to the asphalt pavement (see figure 7.6).

From those results it appears that the thermal regime underneath a block pavement section is more severe than that of asphalt pavement; in terms of both frost depth penetration and of surface minimum temperature. This is also true when comparing a normal concrete pavement with an asphalt pavement. This must be accounted for particularly if the subgrade is frost susceptible and if the water table lies within the range of possible frost penetration.

Those results also indicate that the color of a pavement surface has a greater influence on the thermal regime than the conductivity of the paving materials and the effect of latent heat, i.e. if we assume that block pavement test sections A or D are similar to concrete pavement from a thermal regime standpoint, the block pavement with the granular base has a less severe thermal regime in the frozen state even if the concrete pavement has a thermal conductivity 3 times lower than granular base and that the latent effects of the granular base produced a lower frost penetration and a milder thermal regime.

If the thermal regime of section A is compared to that of

FIGURE 7.1

FROST PENETATION UNDER ASPHALT PAVEMENT

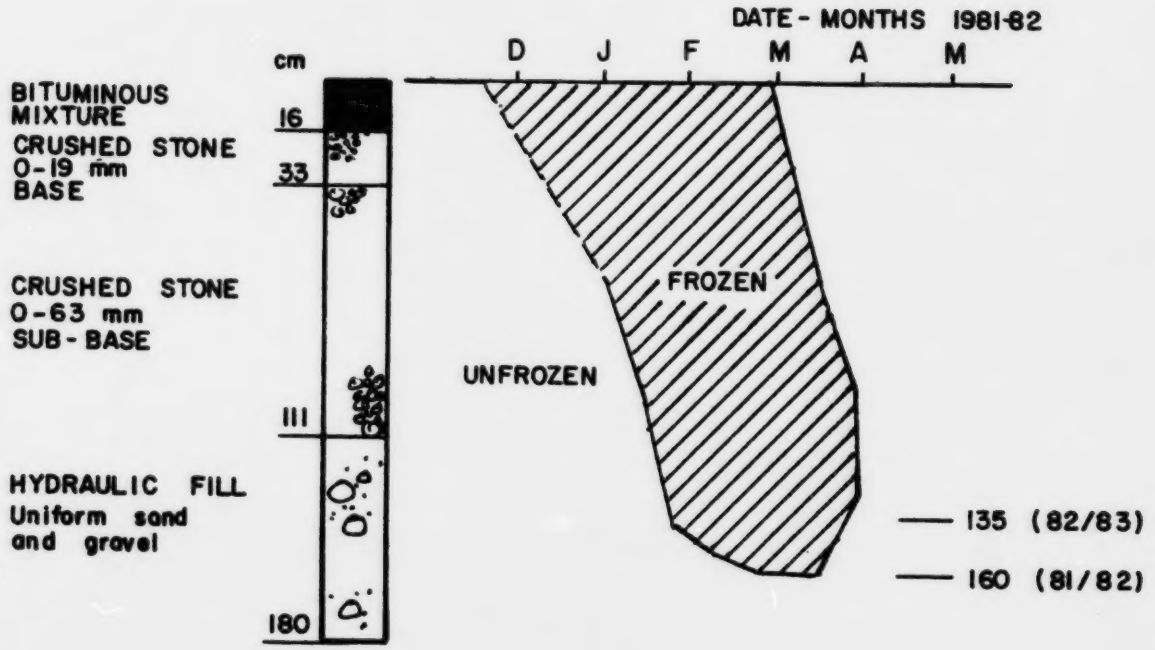


FIGURE 7.2
FROST PENETRATION OF BLOCK PAVEMENT WITH CEMENT-TREATED BASE

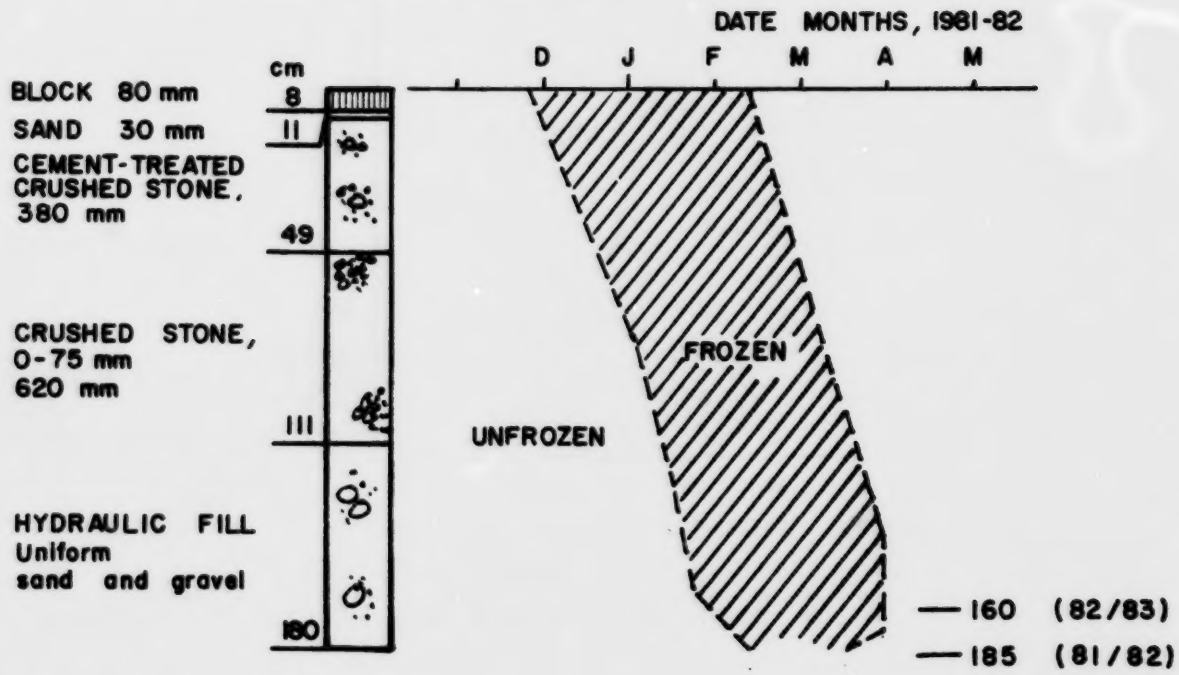


FIGURE 7.3
FROST PENETRATION OF BLOCK PAVEMENT WITH GRANULAR BASE

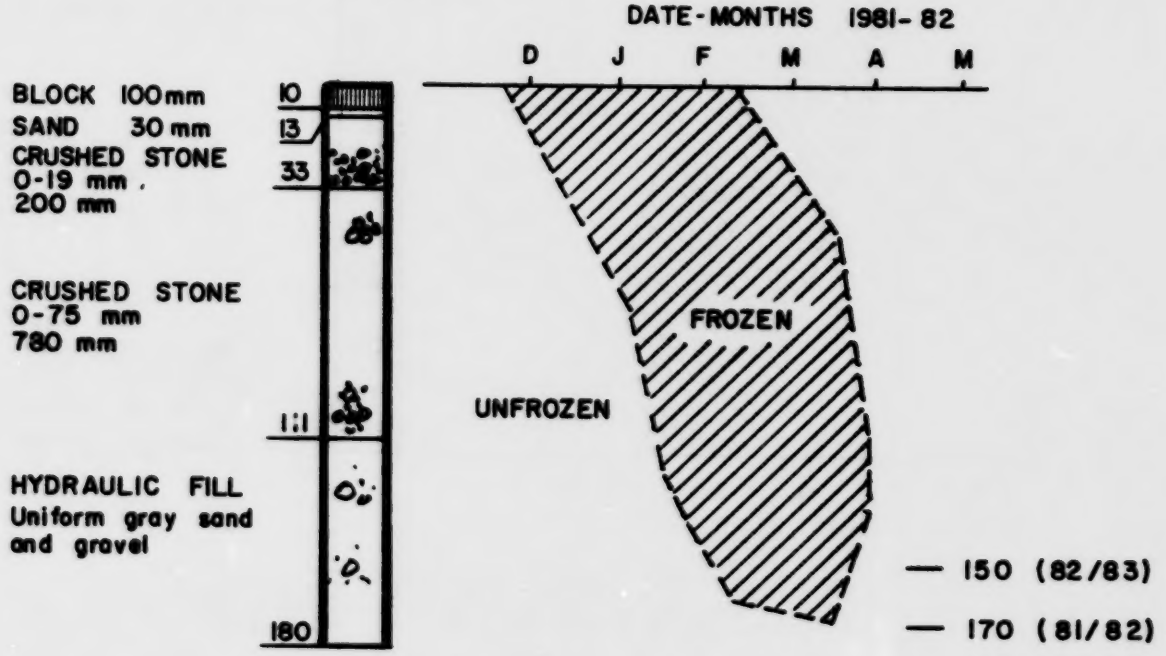


FIGURE 7.4

TEMPERATURE VARIATION UNDER ASPHALT PAVEMENT

WINTER 1981-82

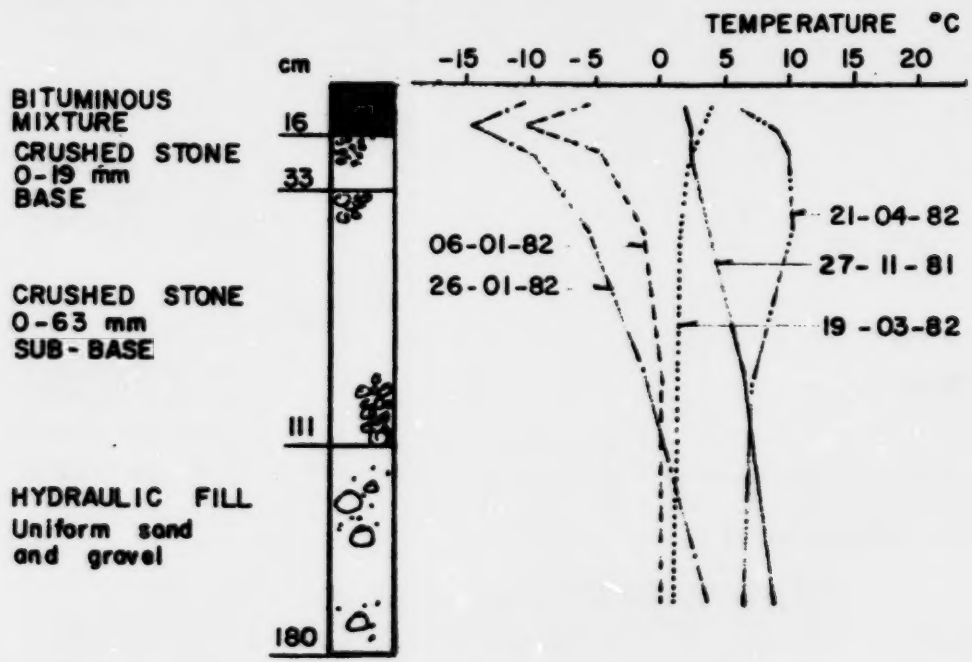


FIGURE 7.5
TEMPERATURE VARIATION UNDER BLOCK PAVEMENT WITH CEMENT STABILIZED BASE - WINTER 1981-82

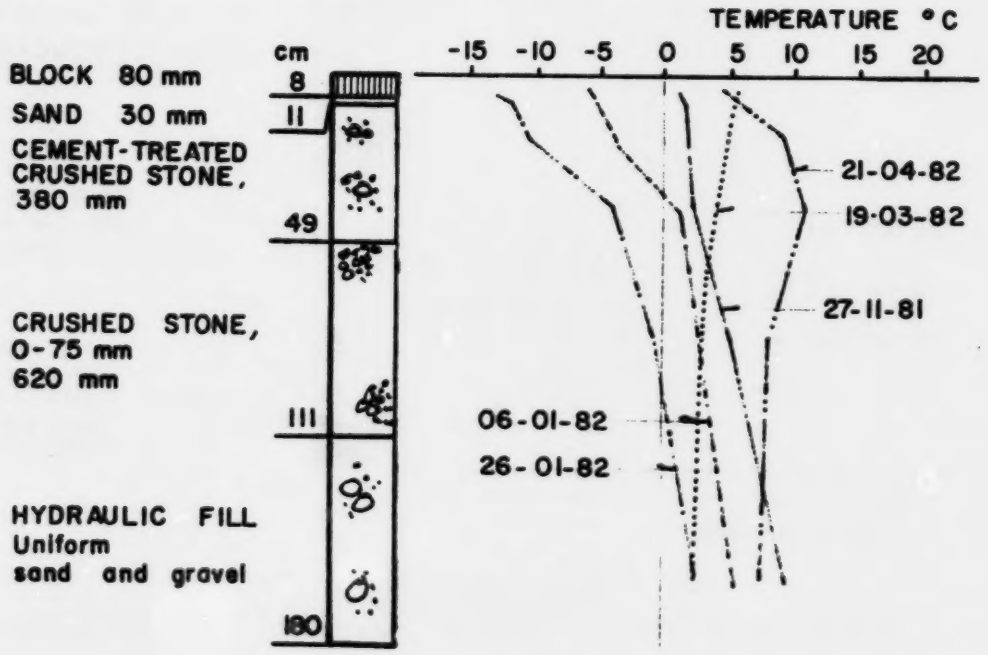
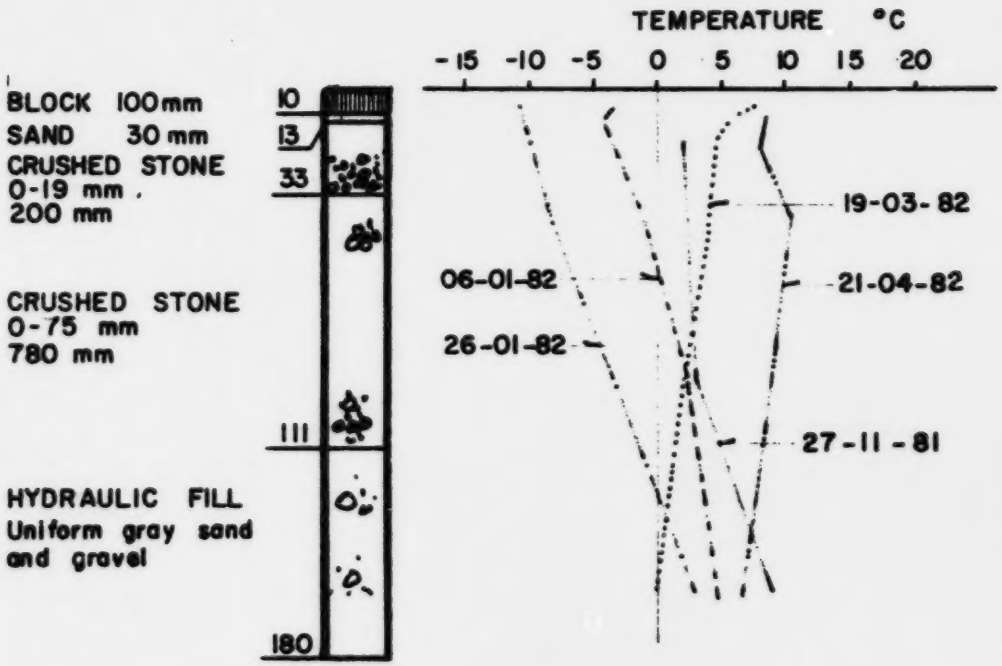


FIGURE 7.6

TEMPERATURE VARIATION UNDER BLOCK PAVEMENT WITH GRANULAR BASE - WINTER 1981-82



section C, it is found that the regime of section A is more severe than that of C, which means that the surface color effect (black (C) versus gray (A)) is even more important than the others effects.

7.2.4 Moisture regime of test sections

Moisture data of pavement layers had been obtained from frozen material during the thawing periods of both winters (1982 and 1983).

Block pavement on granular base showed the following general moisture distribution profiles in the frozen and unfrozen states: the 0-20 mm base course granular material is saturated while the moisture in the 0-75 mm sub-base approaches its residual saturation in the middle of the layer. Suction theory associated with the behavior of granular material while freezing could explain this phenomena. During the thaw period, the block pavement showed an increase in water content due to infiltration of water in the pavement surface and a drainage pattern at the bottom of the 0-75 mm material. During the thawing period moisture contents increased to the end of April as thawing progressed (figure 7.7).

The moisture profile of the base course under the asphalt pavement differs from the block pavement since it is covered by an impervious bituminous concrete. This base course layer is not saturated and the moisture is near the residual water content (figure 7.8).

The difference in moisture content under block pavement, as compared to the asphalt pavement is particularly important during the thaw period, because as with block pavement, water infiltrates through joints, saturates the base course and decreases the bearing capacity.

This difference is also important along side the curb separating block pavement and asphalt pavement because the water which infiltrated along the curb caused as much as 40 mm of settlements of the pavements along the curb, probably by frost heave or hydraulic compactions.

From these results, it appears that it is important to provide the block pavement with a good drainage system or to use a base course material which is insensitive to variation in water content during winter or spring. Also, researches are needed to assess the permeability of block pavement and its variation in time as the joints tighten under dynamic effects of the traffic.

7.3 Structural behavior of test sections

7.3.1 Deflection measurements

Test results show practically no change in deflection with time for block pavements on granular base (figure 7.9), and only a slight decrease of deflection with time for block pavements on

FIGURE 7.7
MOISTURE CONTENT UNDER BLOCK PAVEMENT
WITH GRANULAR BASE

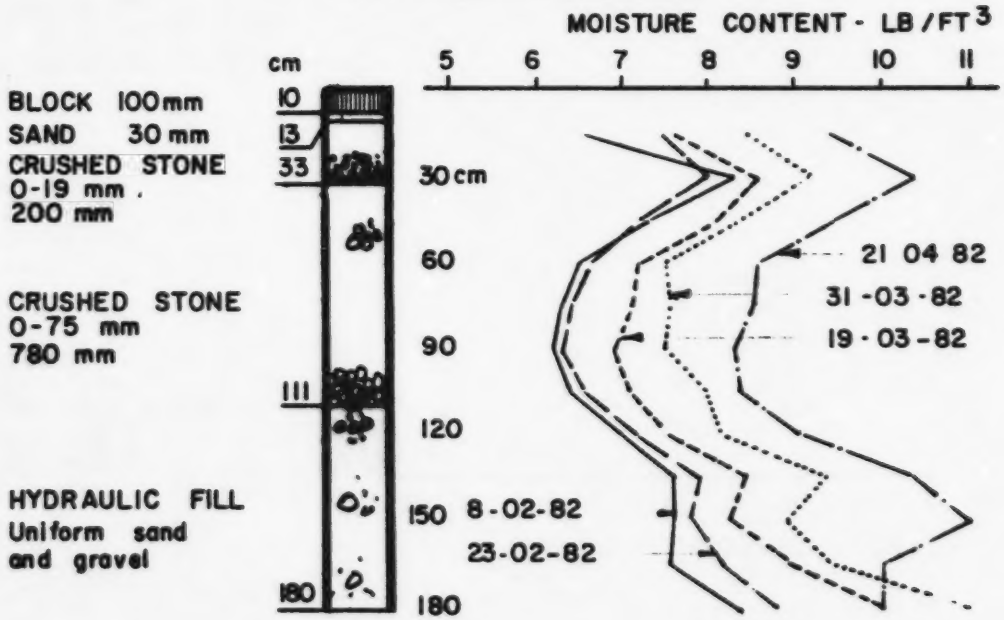
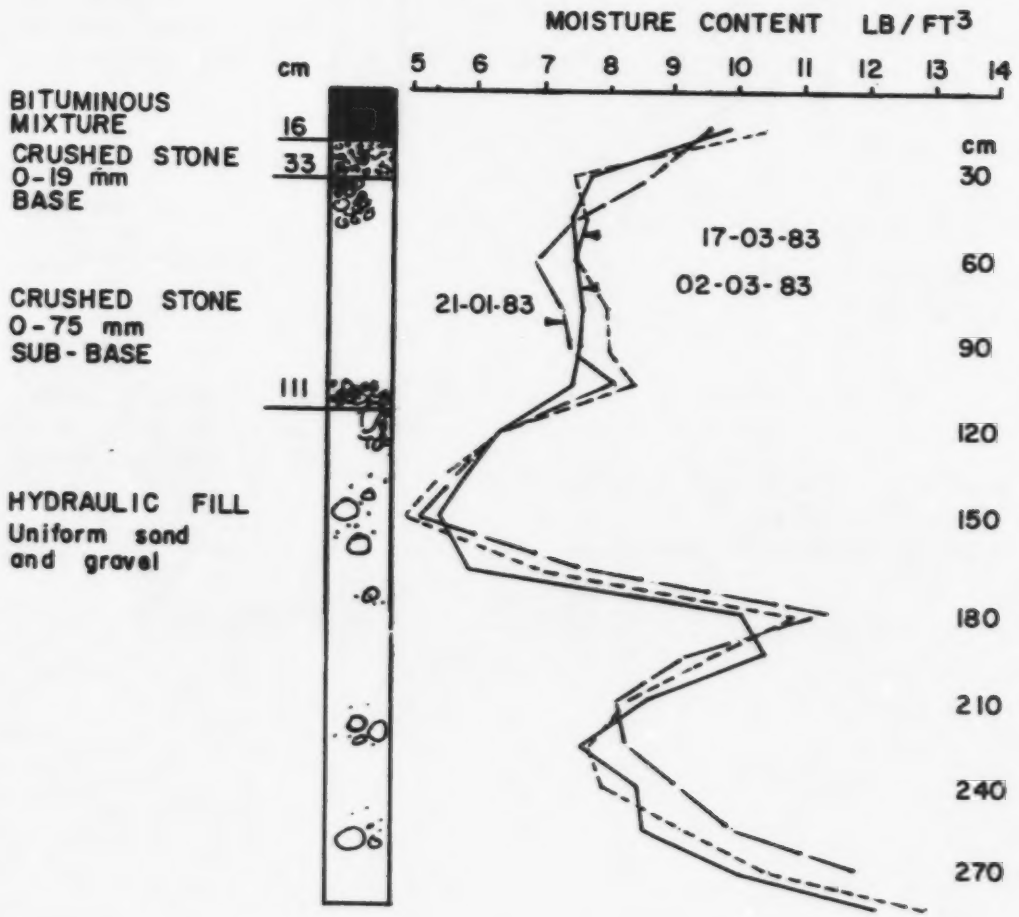


FIGURE 7.8
MOISTURE CONTENT UNDER UNDER ASPHALT PAVEMENT



cement-stabilized base (figure 7.10).

Test results of asphalt pavement are illustrated in figures 7,11 and 7,12 for three distinct areas in the container port: the test section C, and an old pavement section on figure 7.11 and the pavement between the rails of the gantry crane (figure 7.12). All three sections show strengthening of pavement with time, by a decrease in maximum deflection of almost 30%. The difference in the behavior of block and asphalt pavements may be attributed to the moisture content of the 0-20 mm crushed stone bases: having increased under the paving blocks, it decreased to the residual moisture content under the asphalt pavement.

Deflections were also measured with Benkelman beam for conditions of heavy loading using a gantry crane with wheel loads of 45 tons. Test results are summarized in Table 7.1 for block pavements on cement-stabilized and granular bases and asphalt pavement on a granular base. The results clearly indicate that, under very heavy wheel loads, a block pavement with granular base performs as a flexible pavement whereas block laid on a good, cement-stabilized crushed stone base performs as a rigid pavement with a very low surface deflection and a very high spreadability index.

7.3.2 Degradation of pavements

At the Racine Terminal, two years after construction, the block pavement (section A and B) with granular base is still in excellent condition (figure 7.13). Very few blocks are cracked and there is only one localized minor surface depression near the edge of the circulating area noted in september 1982 but which had disappeared in spring of 1983.

The performance of block pavement with cement stabilized base (section D and E) is in equally good condition after two years of service (figure 714) except in one area (in section E) where blocks cracked under containers legs.

The performance of the asphalt pavement was also excellent except for a few low temperature cracks.

The only serious problem is caused by water infiltration along the perimeter curb of the block pavement. Indeed, where block or asphalt rest on a granular base, the pavement edges settled as much as 40 mm relative to the curb level. These settlements are the cause of longitudinal edge-cracking in the asphalt surface and are generally more pronounced at the block side of the curb than at the asphalt side. In the fall of 1982, one year after construction, depressions along the perimeter curb were corrected by raising the base of the block pavement and by patching the asphalt pavement.

In contrast to the block pavement at the Racine Terminal, the Task Terminal block pavement has not performed well. With but one year of service, the pavement failed completely under the action of the crawler crane along the dockside (figure 7.15). Because the quality of the blocks and the base course compaction

FIGURE 7.9

CHANGES IN STRUCTURAL BEHAVIOUR OF BLOCK PAVEMENT WITH TIME

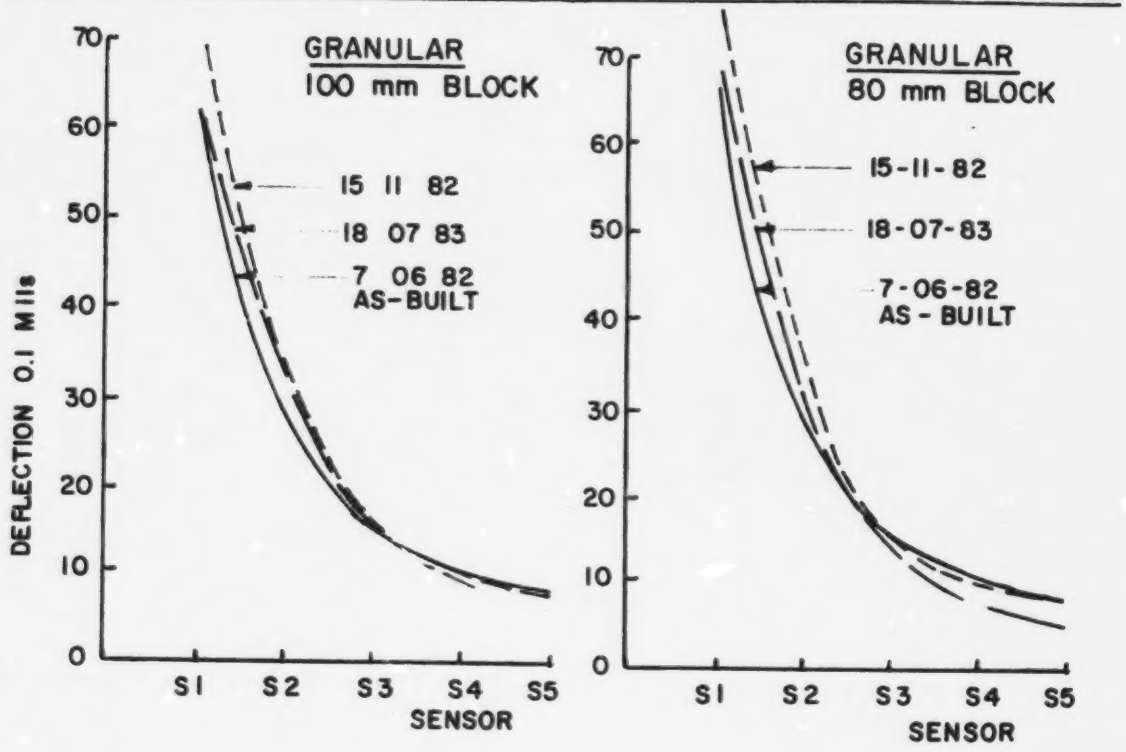


FIGURE 7.10

CHANGE IN STRUCTURAL BEHAVIOR OF BLOCK PAVEMENT WITH TIME

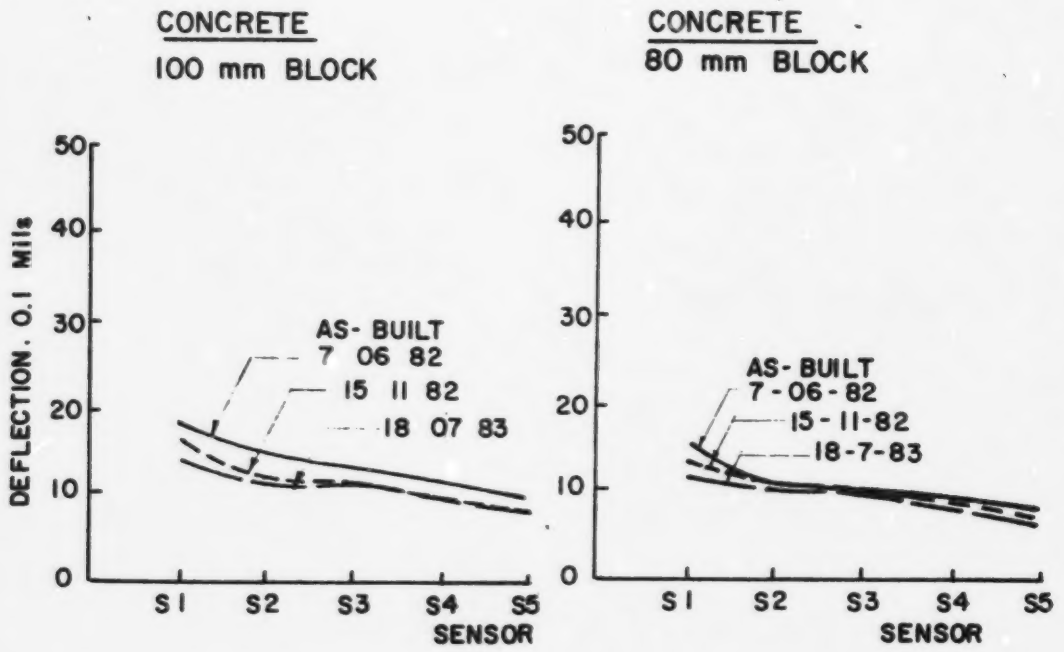


FIGURE 7.11
CHANGES IN STUCTURAL BEHAVIOUR
OF ASPHALT PAVEMENT

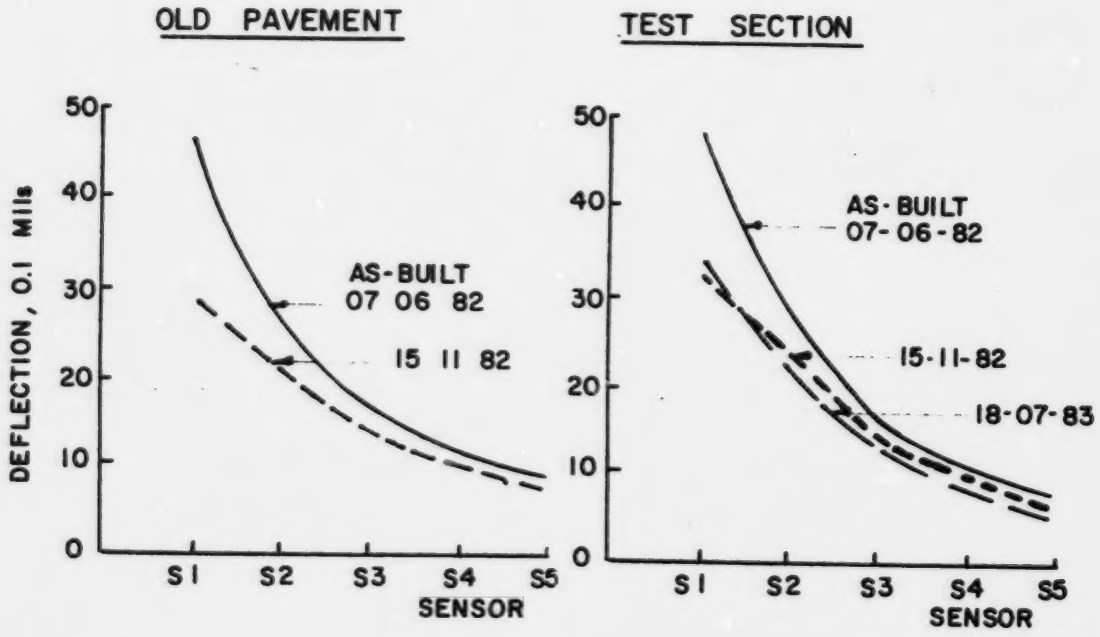


FIGURE 7.12
CHANGES IN STRUCTURAL BEHAVIOR OF ASPHALT PAVEMENT

BETWEEN RAILS
(GANTRY)

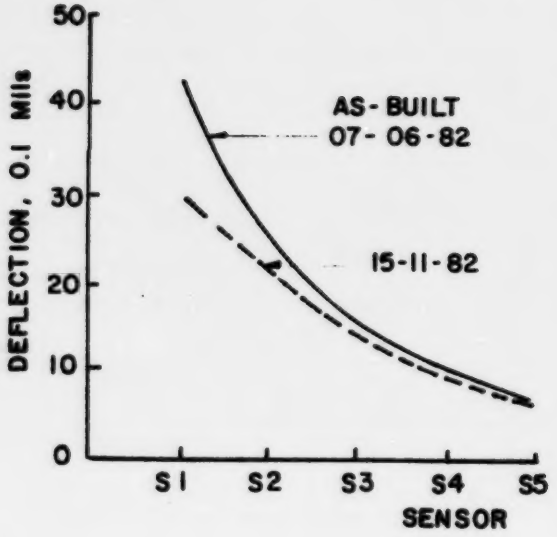


Table 7.1

DEFLECTION MEASUREMENTS UNDER DYNAFLECT AND GANTRY LOADINGS

TYPE OF PAVEMENT	BLOCK PAVEMENT		15 mm ASPHALT ON GRANULAR BASE
	Cement-stabilized	Granular base	
Under Dynaflect Loading			
Max. deflection, in.	0.0012	0.0065	0.0035
Surface curvature, in.	0.0002	0.0031	0.0011
Spreadability Index, %	80	40	50
Under Gantry Loading			
Max. delfection, in.	0.0040	0.0410	0.0300
Surface curvature, in.	0	0.0140	0.0090
Spreadability Index, %	100	47	51

FIGURE 7.13
PAVEMENT CONDITION SURVEY AT RACINE TERMINAL
BLOCK PAVEMENT ON GRANULAR BASE

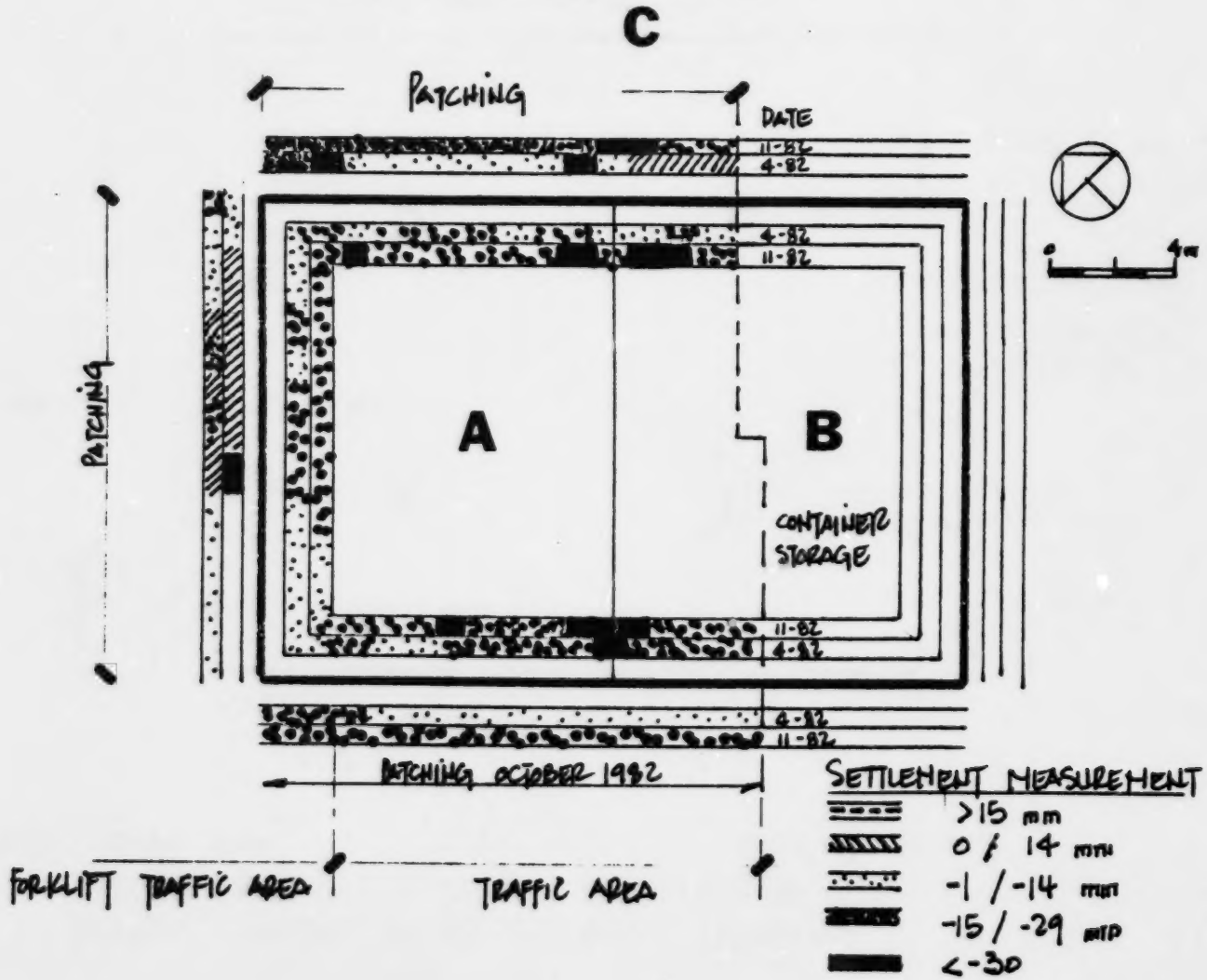


FIGURE 7.14

PAVEMENT CONDITION SURVEY AT RACINE TERMINAL

BLOCK PAVEMENT ON CEMENT-STABILIZED BASE

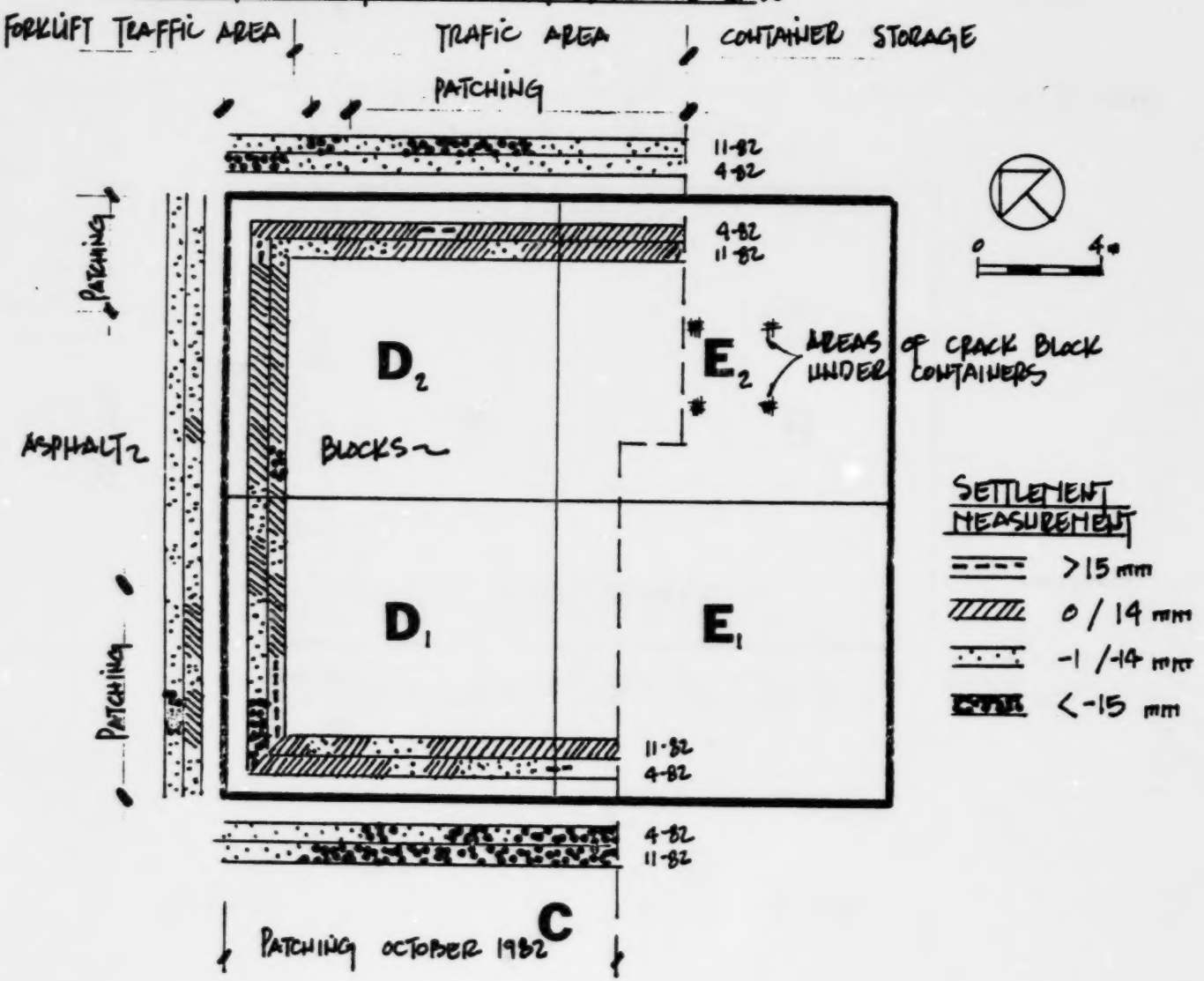
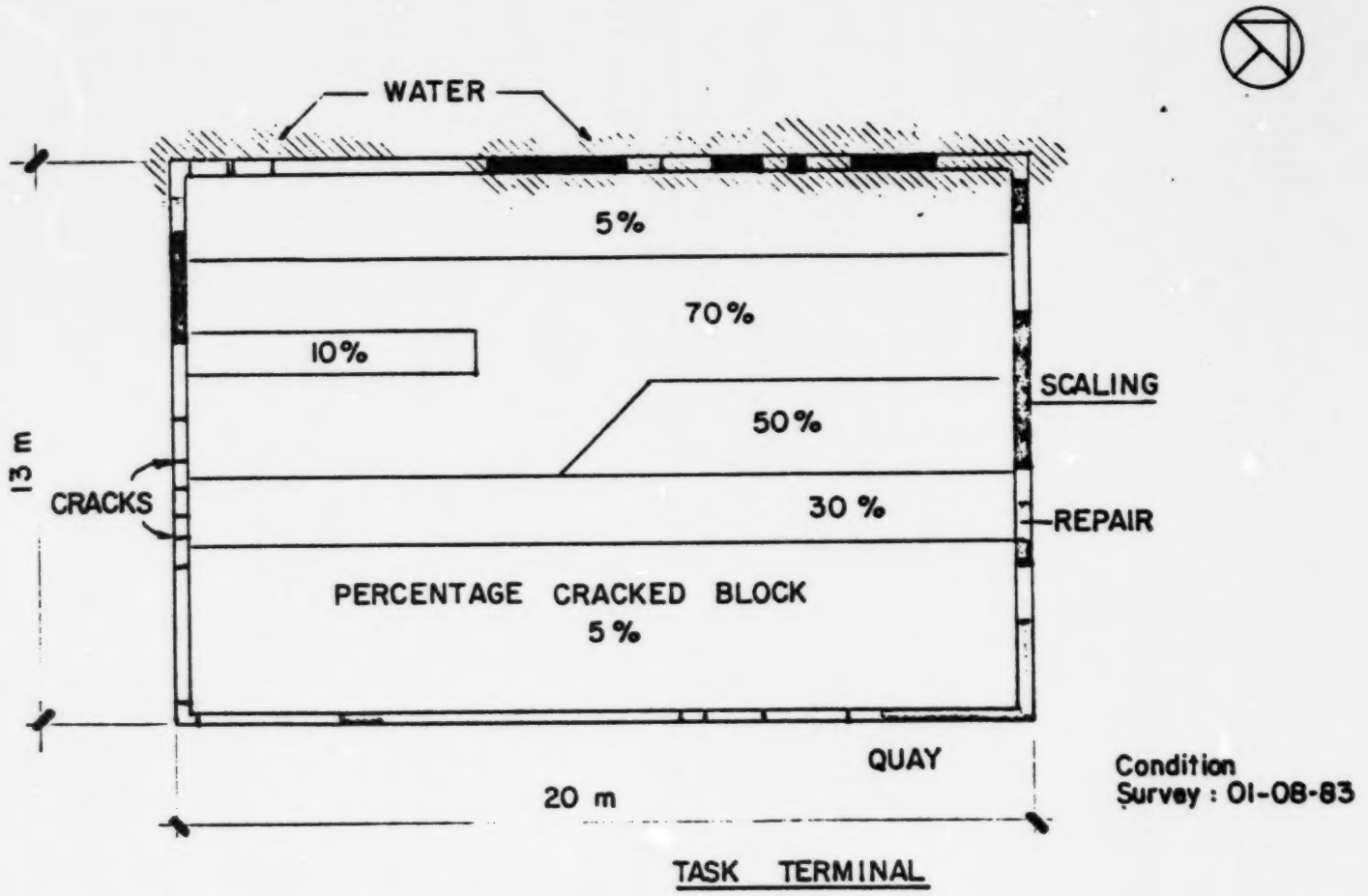


FIGURE 7.15
PAVEMENT CONDITION SURVEY AT
TASK TERMINAL (UNDER CRAWLER CRANE)



were questioned, the pavement was rebuilt with new block and a new base to rigid specifications and control. Although the new pavement has performed better, it too, is degrading rapidly. This experience indicates that interlocking paving blocks should not be used under such severe loading conditions.

7.4 Preliminary conclusions on actual performance and future research needs

1. Six test sections have been designed and built to compare:

a) 80 and 100 mm block pavements with 150 mm thick asphalt pavement

b) two types of foundation under block pavements: cement treated granular crushed stone base

c) the effects of loading vehicles on performance: rubber-tired gantries with crawler cranes.

2. The pavement structure of the five sections at the Racine Terminal have been adequately designed by the USCE Method for Flexible Pavement and the PCA Method for Cement-Stabilized Base Pavement. All construction materials were of high quality and closely controlled.

3. The performance of pavements, during and after construction, were closely monitored by measuring the strength of each component layer, the change in moisture condition, the temperature variation, the frost penetration and the rutting and cracking of surfaces.

4. Typical values of Dynamic Moduli of different paving materials have been established in terms of mean and dispersion as well as the gain in strength and spreadability of pavement with the addition of each layer.

5. Proof-rolling by several passes of a 30-T rubber-tired roller has no effect on a block pavement.

6. The pavements have gone through two winters:one severe and one mild. Frost penetration varied between 1,38 to 1,88 meters, with the highest penetration in the block pavement with concrete base and the lowest under the asphalt pavement. From a thermal point of view, a block pavement could considered to be the same as a concrete pavement.

7. AFter two years of service, the moisture content of the 0-19 mm base course is saturated under block pavement and near its residual saturation under asphalt pavement, indicating the need for more drainage under a block pavement with a non-treated granular base.

8. The asphalt pavement gained strength during the first year of operation while the strength of a concrete block pavement remained constant.

9. Under gantry loading, block pavement with granular base performs the same as a flexible pavement whereas block pavement with a cement-stabilized crushed stone base performs as a rigid pavement.

10. Both block pavements (as designed) performed well after two years of traffic composed of trucks, trailers, fork lifts, and gantries or straddle carriers. The pavement with granular base appears more sensitive to drainage whereas the pavement with cement-stabilized base appears more sensitive to concentrated loads. More study is needed in this area.

11. Pavement sections made with either 100 mm or 80 mm blocks have given equal performance, after two years of service.

12. More research is needed to clearly define the limitations of use in areas where dockside crawler cranes or containers may damage blocks.

13. Drainage should be provided along curbs to control settlement and heaving.

14. The present cost of concrete pavement is significantly higher than asphalt pavement. However, it will become competitive as the initial cost of block surfacing approaches that of asphalt surfacing or as the additional benefits of block pavement are proven (resistance to cracking, greater flexibility, resistance to oil).

ANNEX SECTION 7

FIELD TEMPERATURE AND MOISTURE DATA

ANNEX TO SECTION 7

TEST SECTION A (block pavement untreated granular base)

Frost depth penetration	1981-1982
Temperature profile	1981-1982
Temperature profile	Spring 1983
Moisture profile	Spring 1982
Moisture profile	Spring 1983

TEST SECTION C (asphalt pavement)

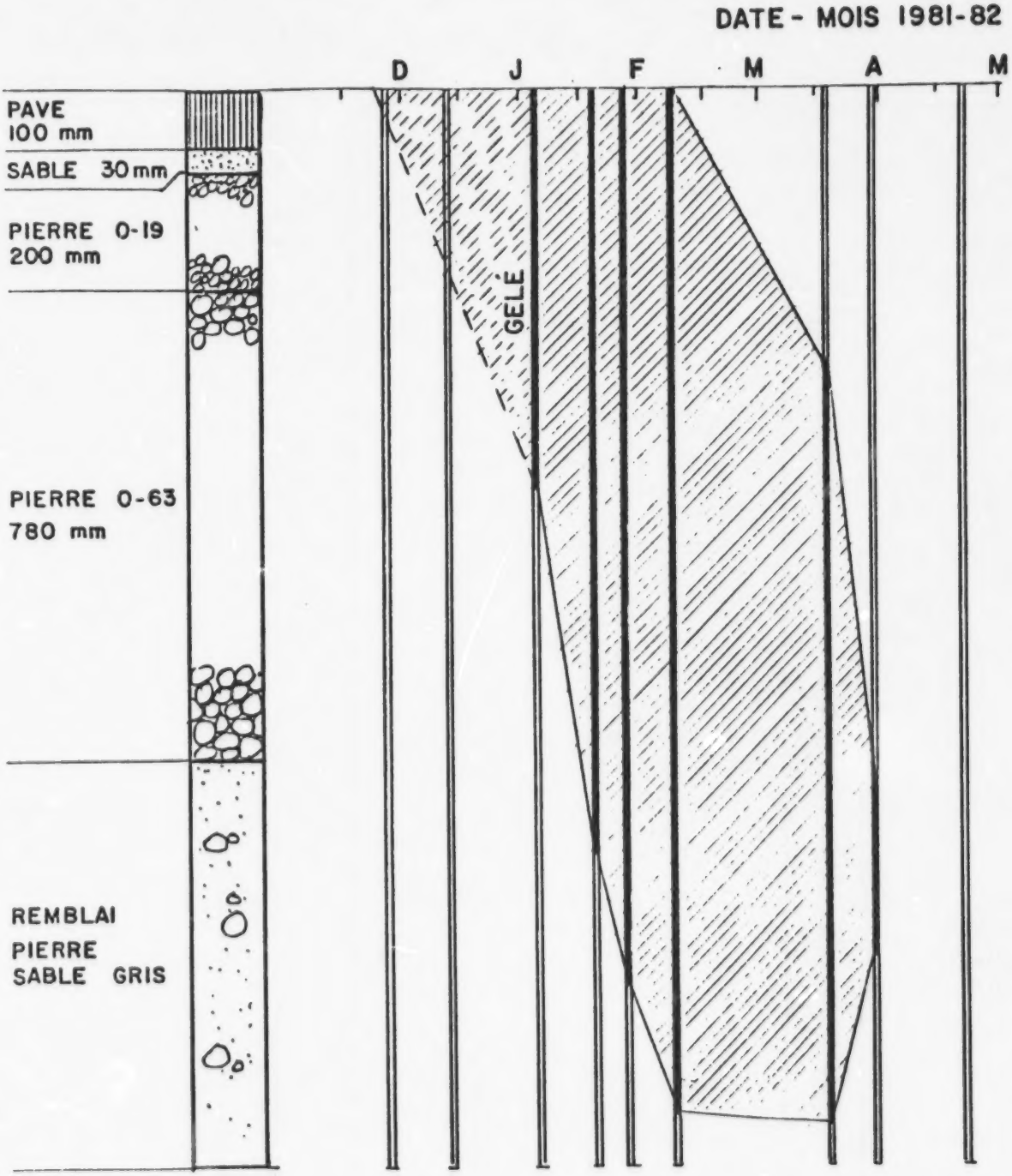
Frost depth penetration	1981-1982
Frost depth penetration	1982-1983
Temperature profile	1981-1982
Temperature profile	Spring 1983
Moisture profile	November 1982
Moisture profile	Spring 1983

TEST SECTION D (block pavement, cement treated granular base)

Frost depth penetration	1981-1982
Frost depth penetration	1982-1983
Temperature profile	1981-1982
Temperature profile	1983
Moisture profile	Spring 1982
Moisture profile	Spring 1983

VARIATION DE LA PENÉTRATION DU GEL

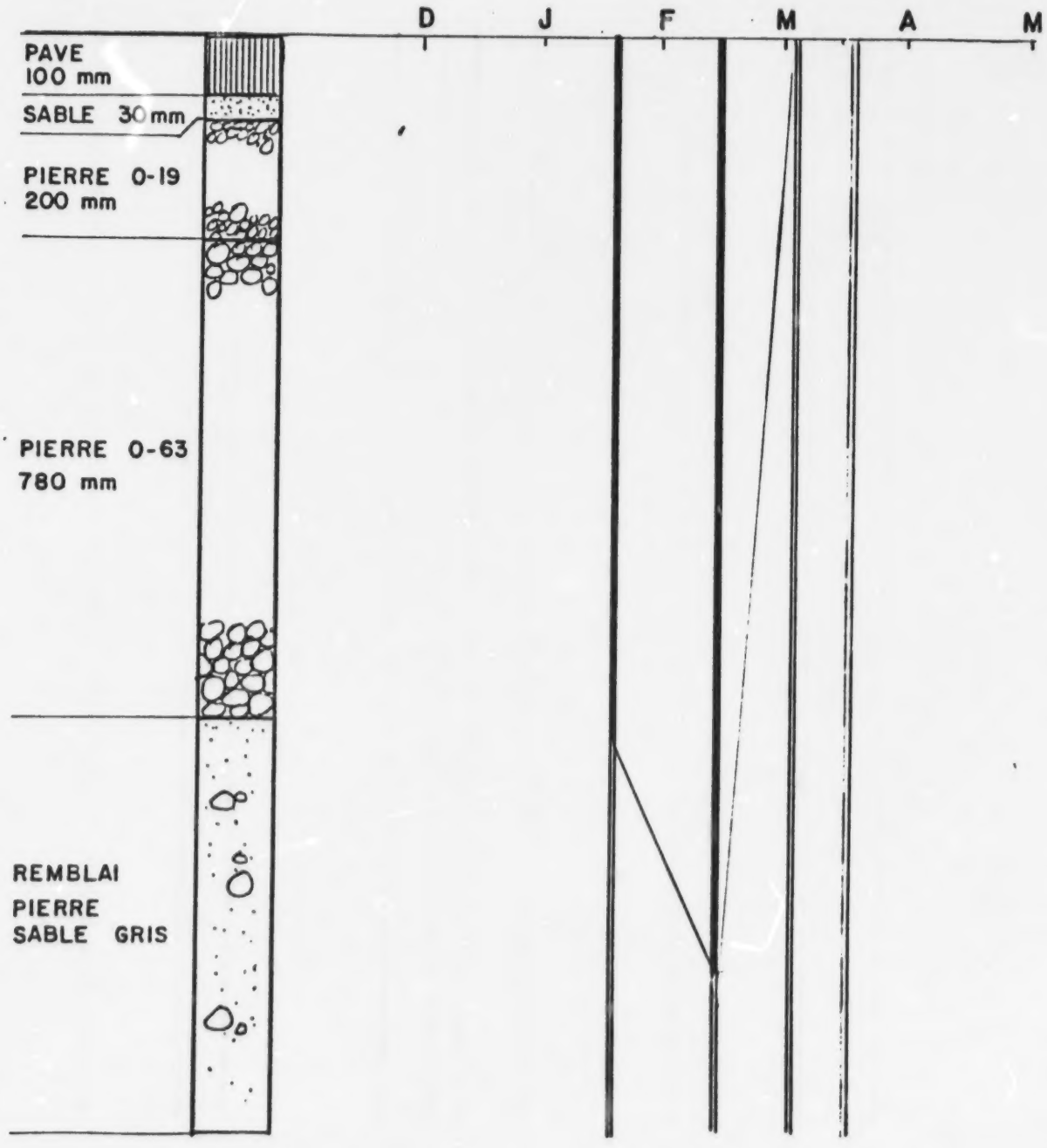
DALLE A



VARIATION DE LA PENÉTRATION DU GEL

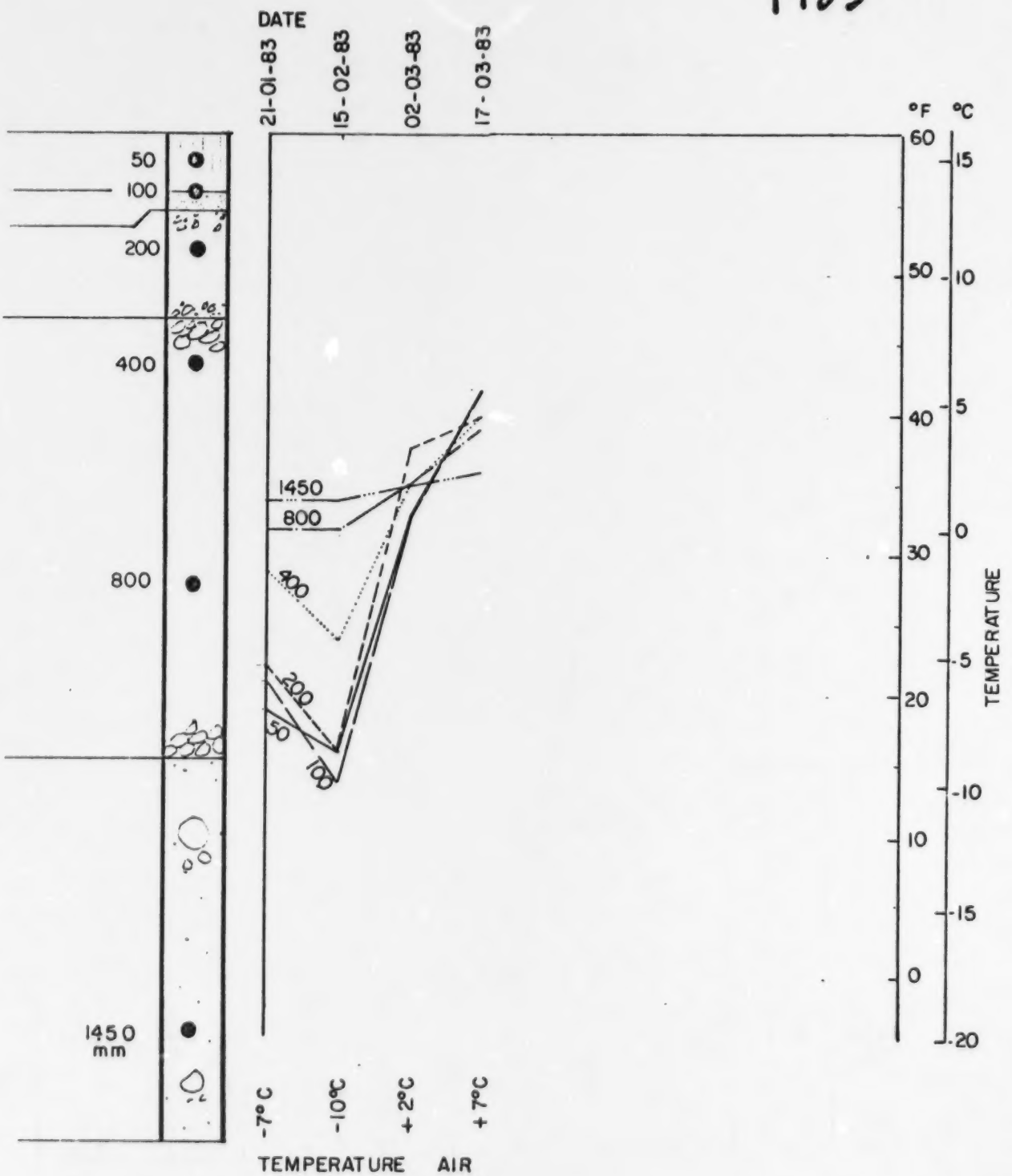
DALLE A

1983



VARIATION DE TEMPÉRATURE

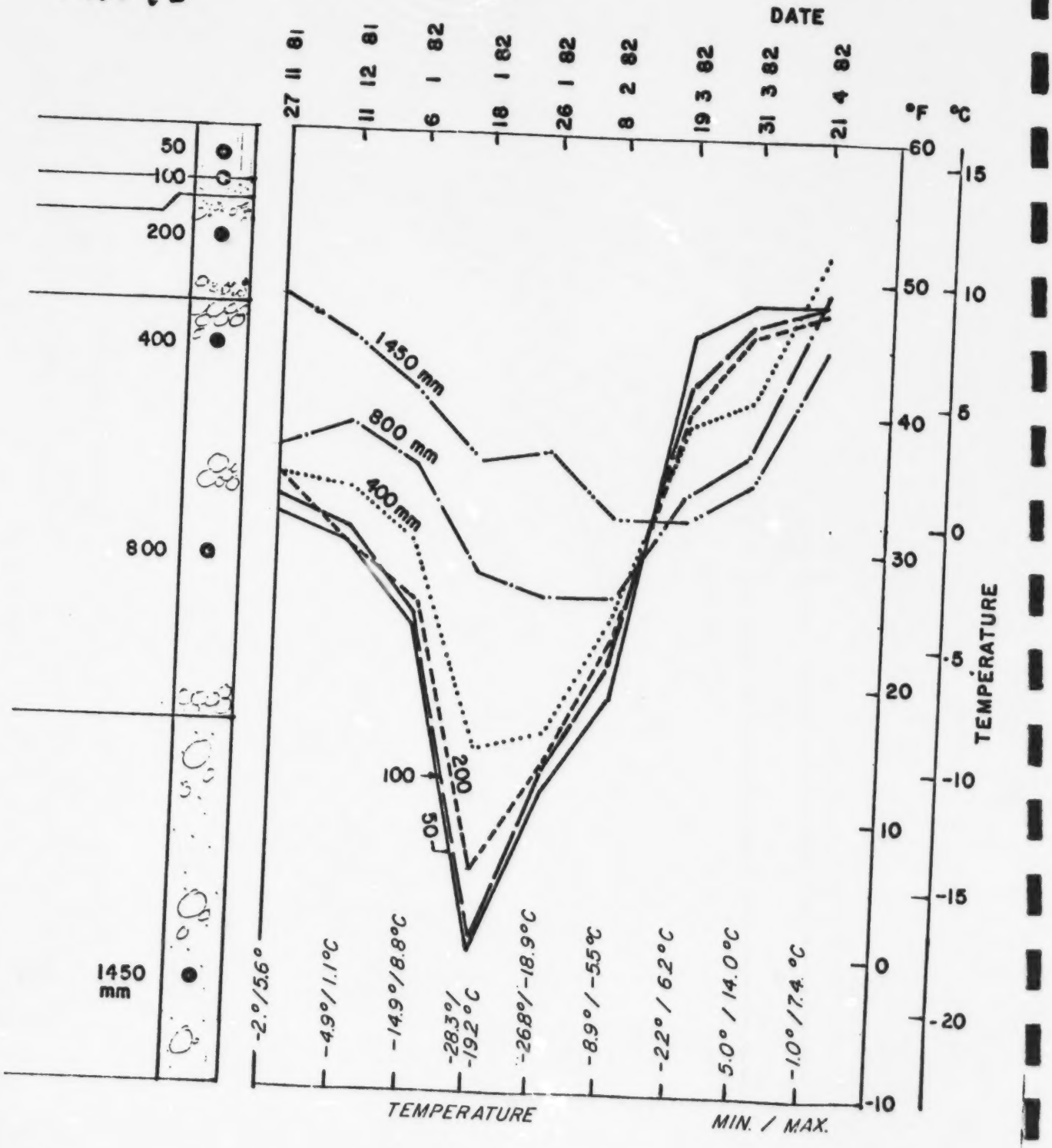
DALLE A
1983



VARIATION DE TEMPÉRATURE

1981-82

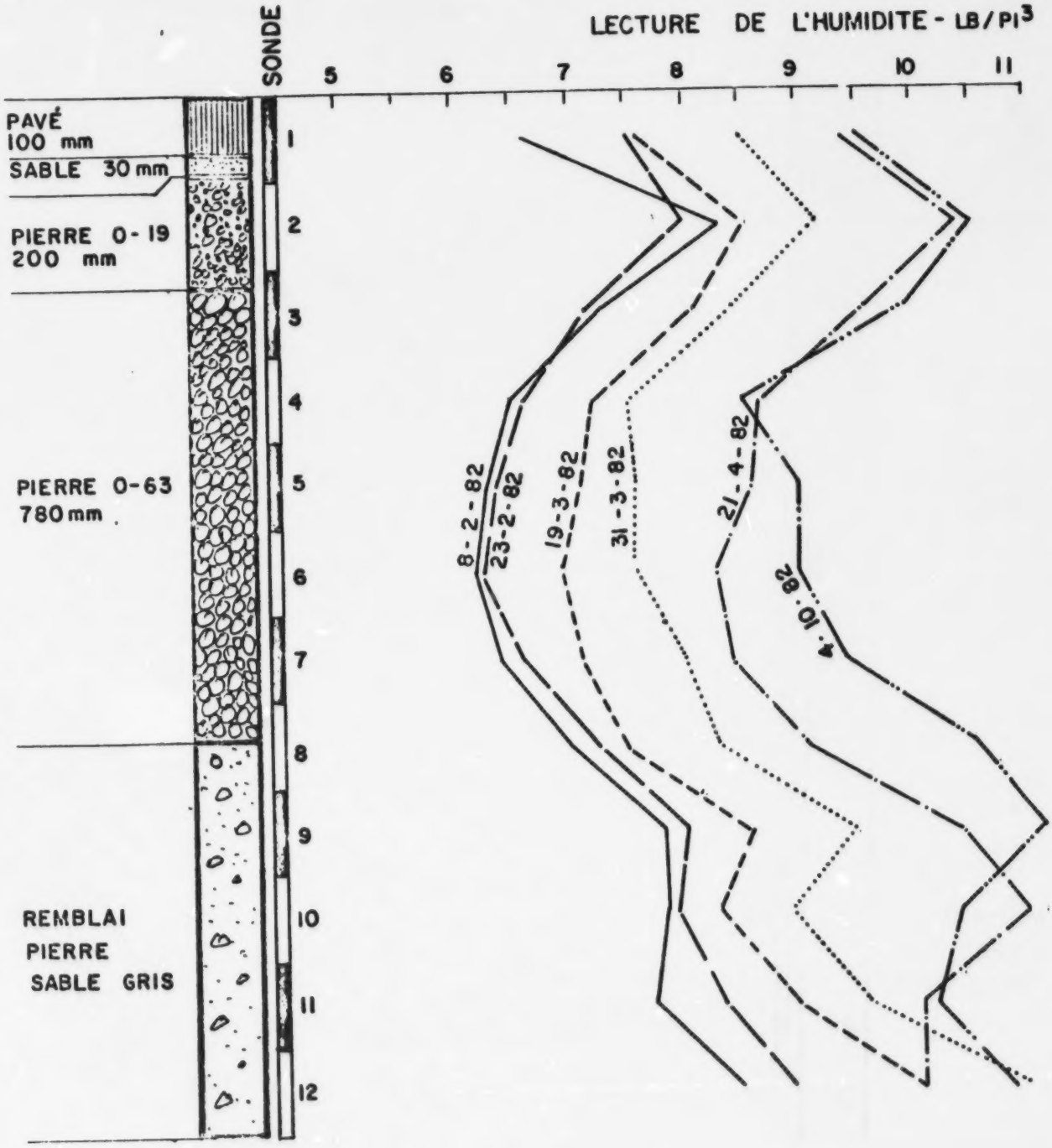
DALLE A



VARIATION DE L'HUMIDITÉ

DALLE A

1982



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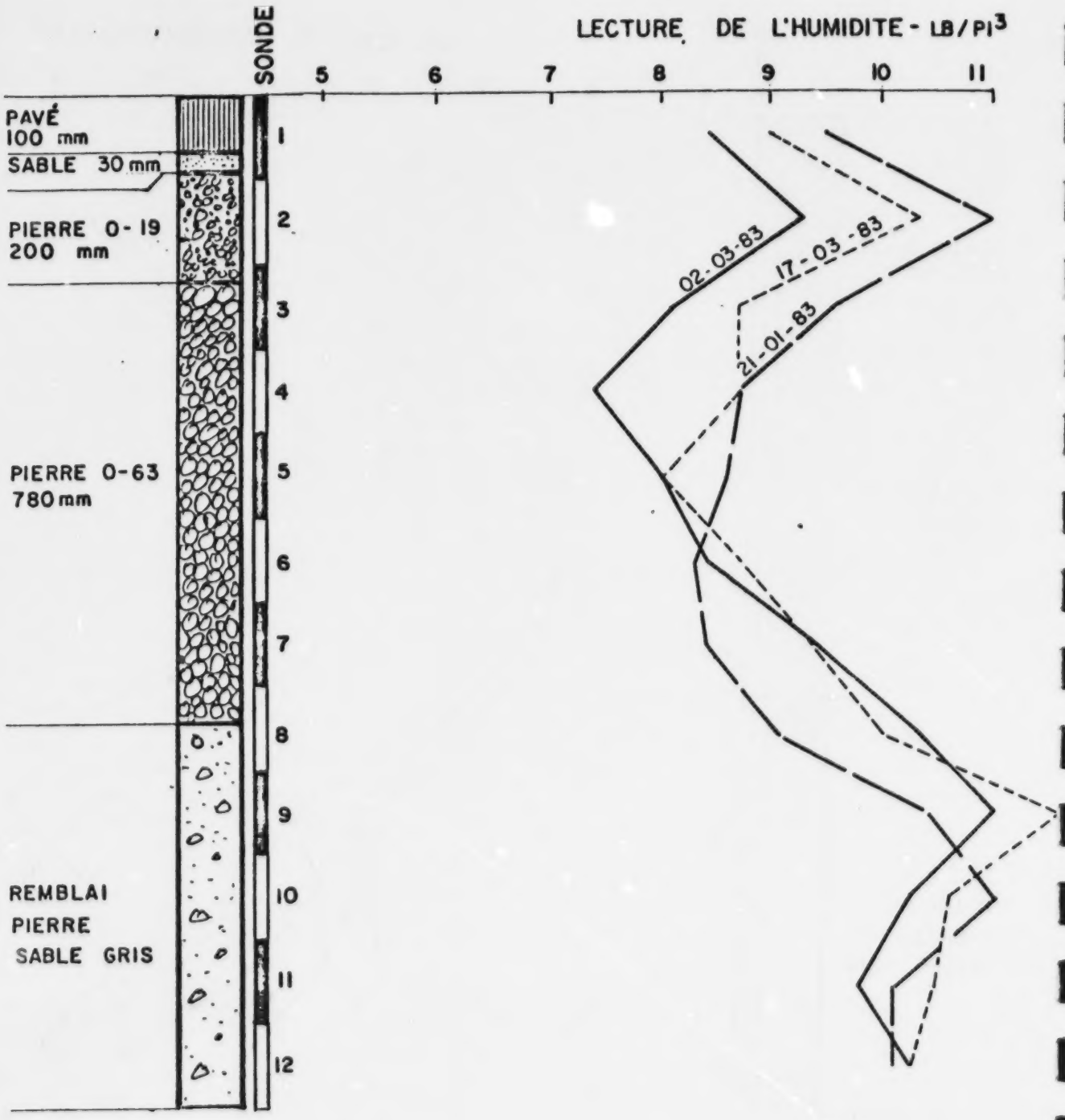

VARIATION DE LA PÉNÉTRATION DU GEL

DALLE C
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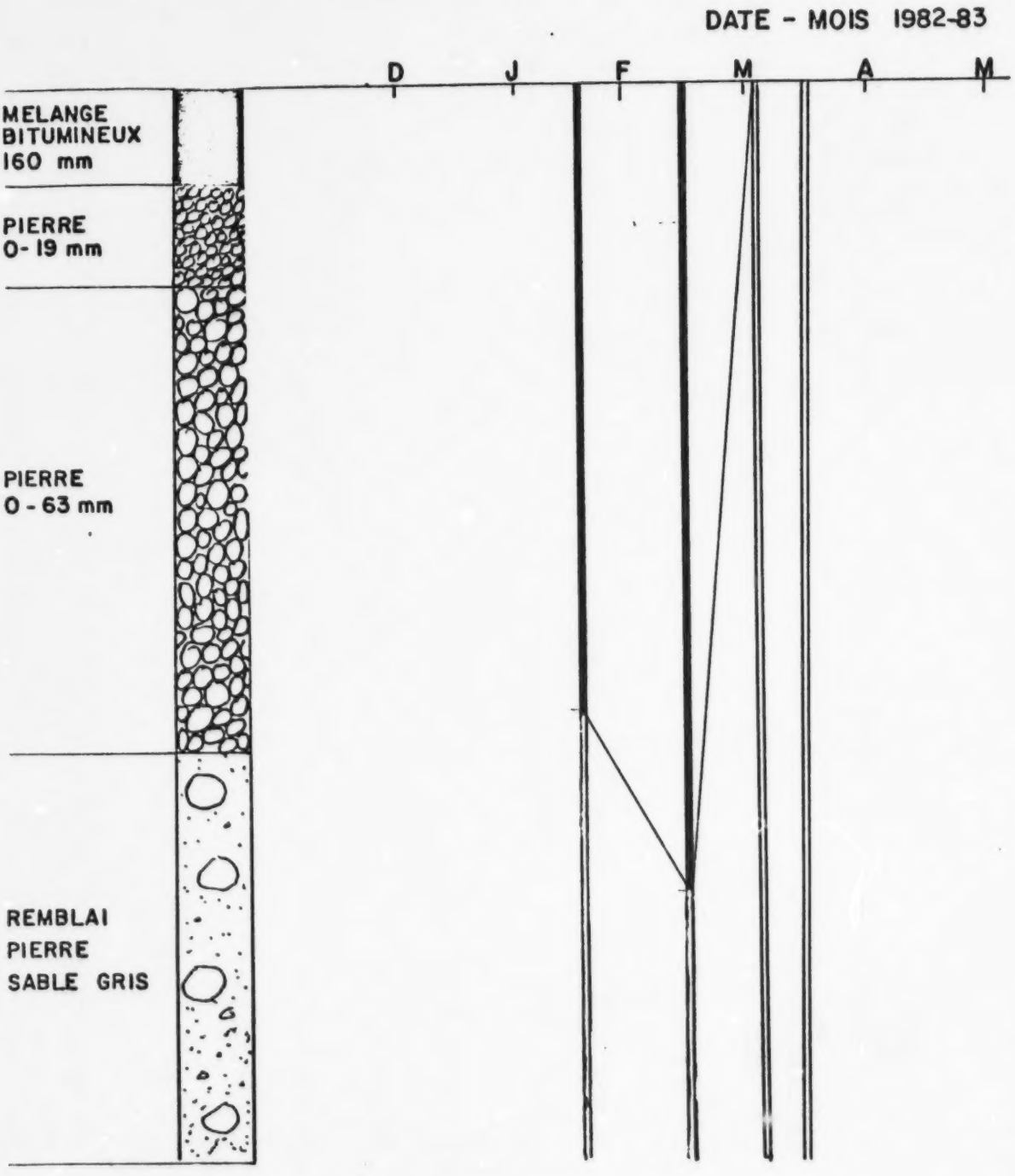
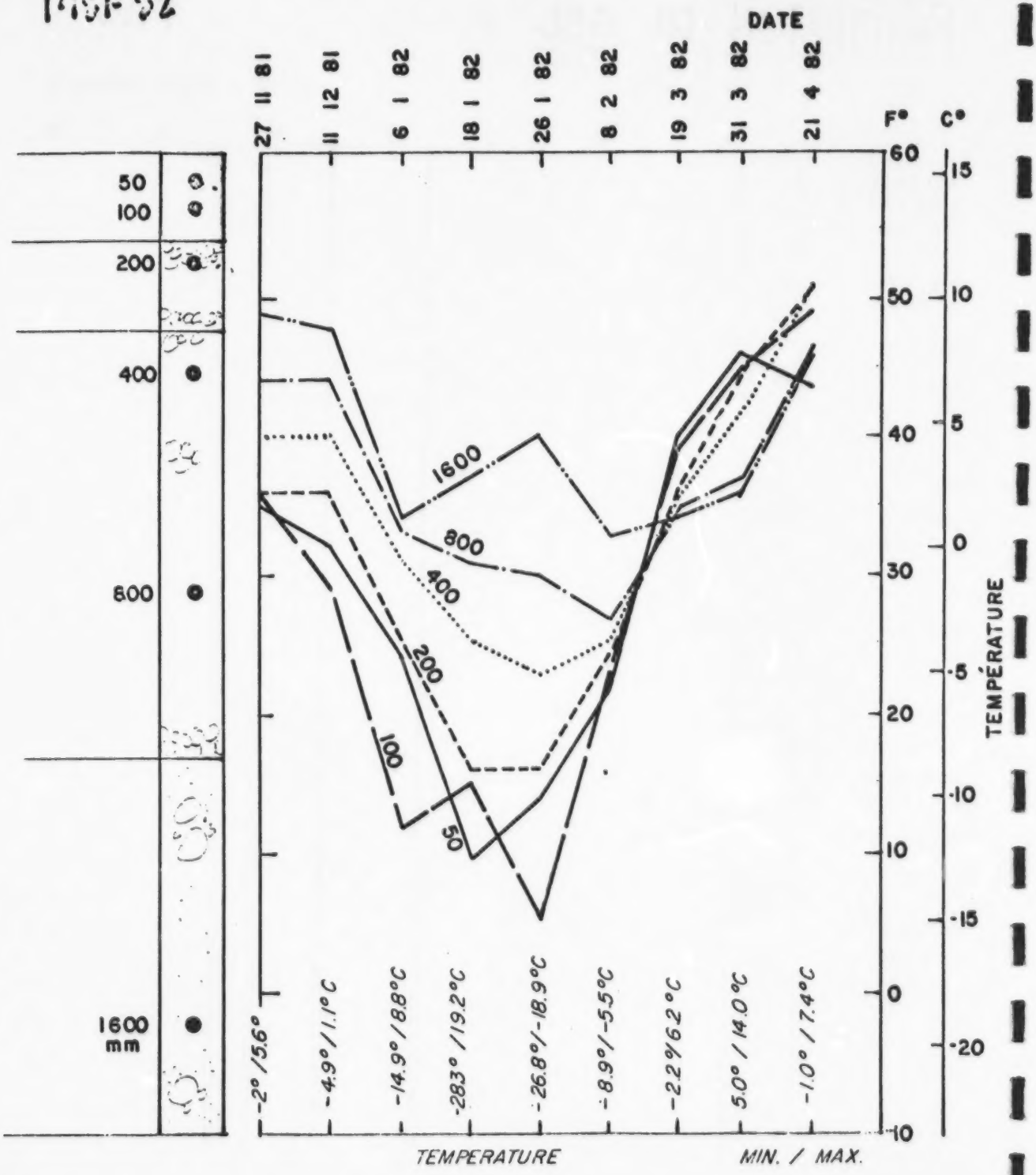


VARIATION DE TEMPÉRATURE — DALLE C

1981-82



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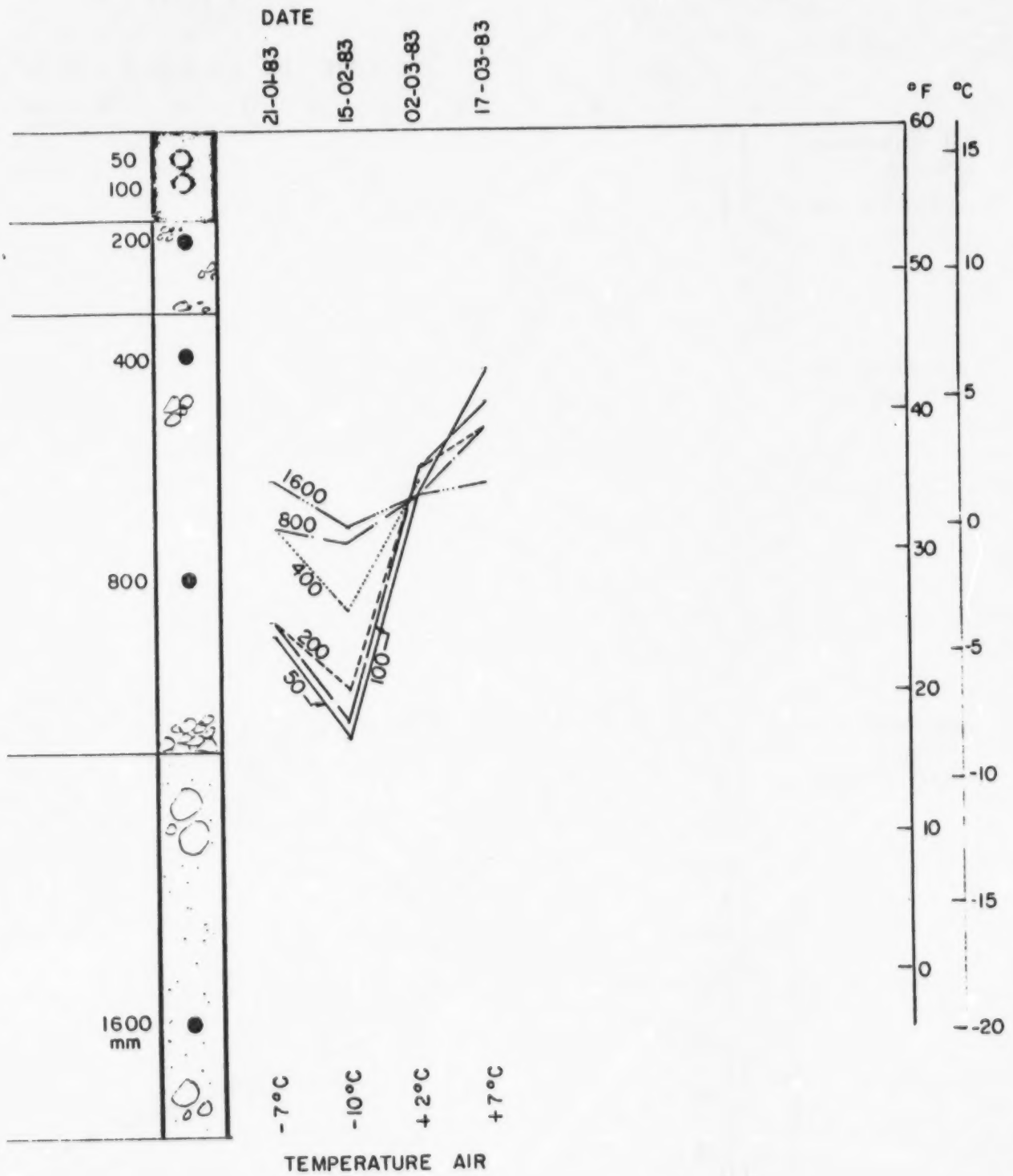

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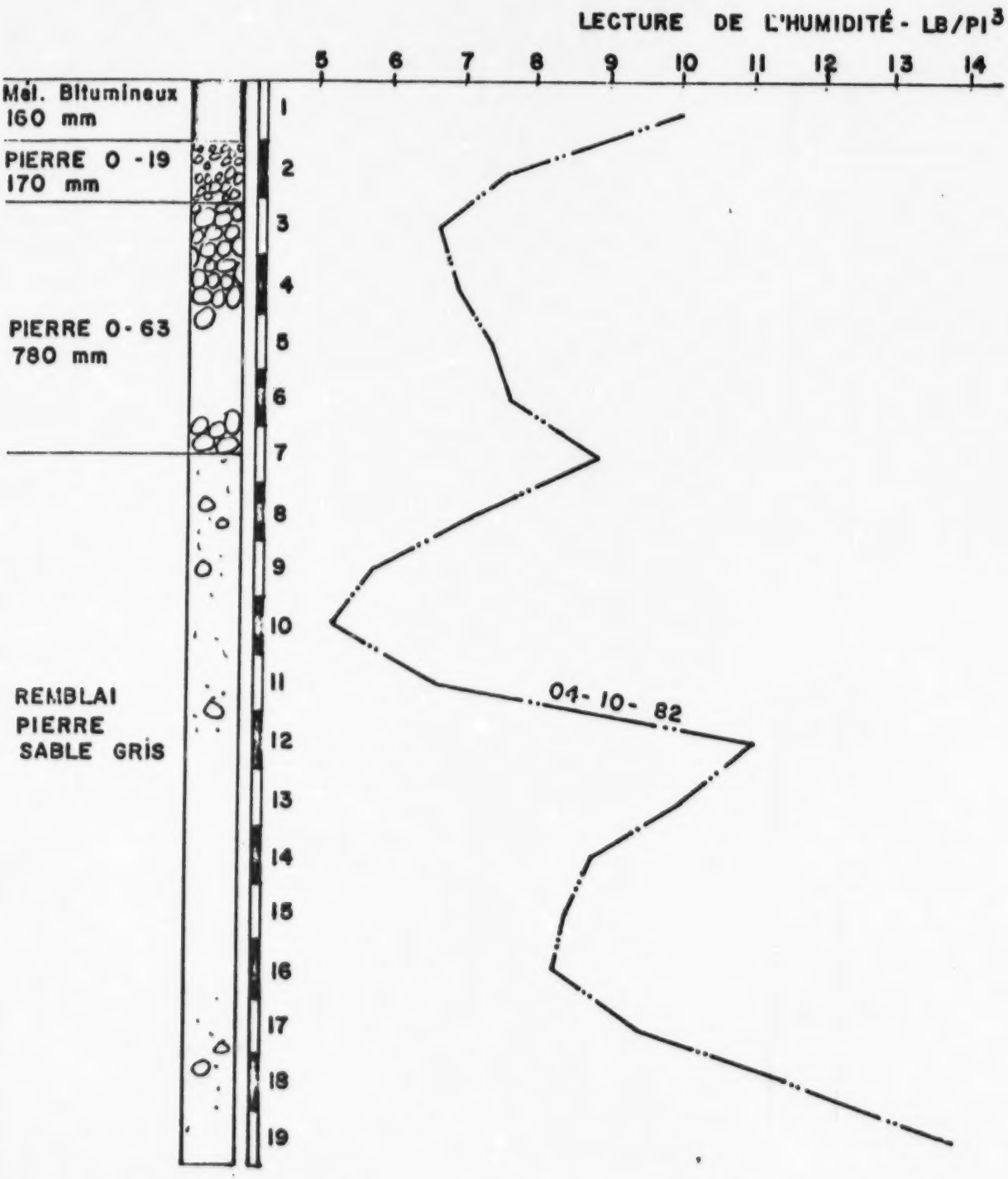
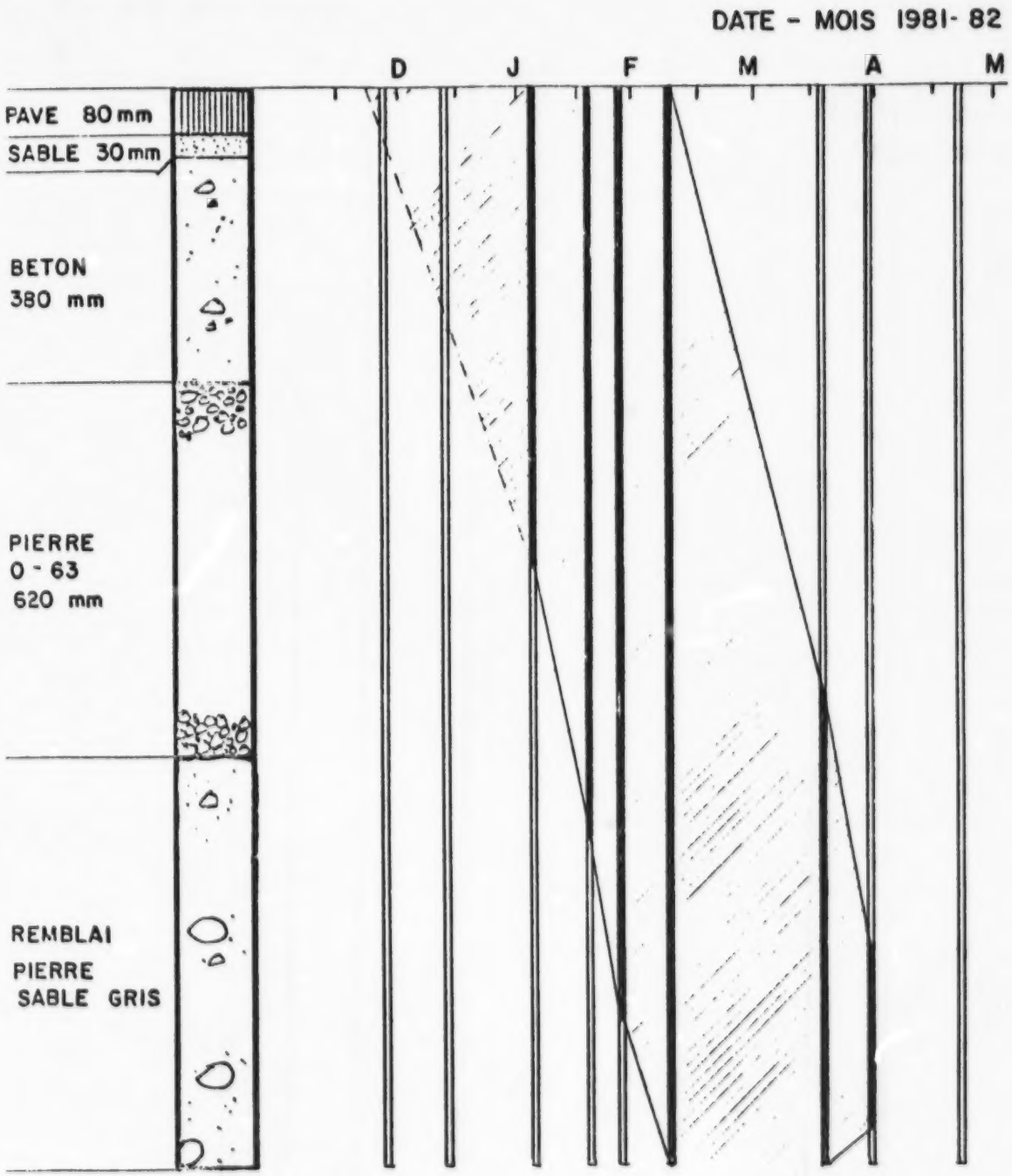
DALLE C

1981-82



VARIATION DE LA PÉNÉTRATION DU GEL

DALLE D
1982



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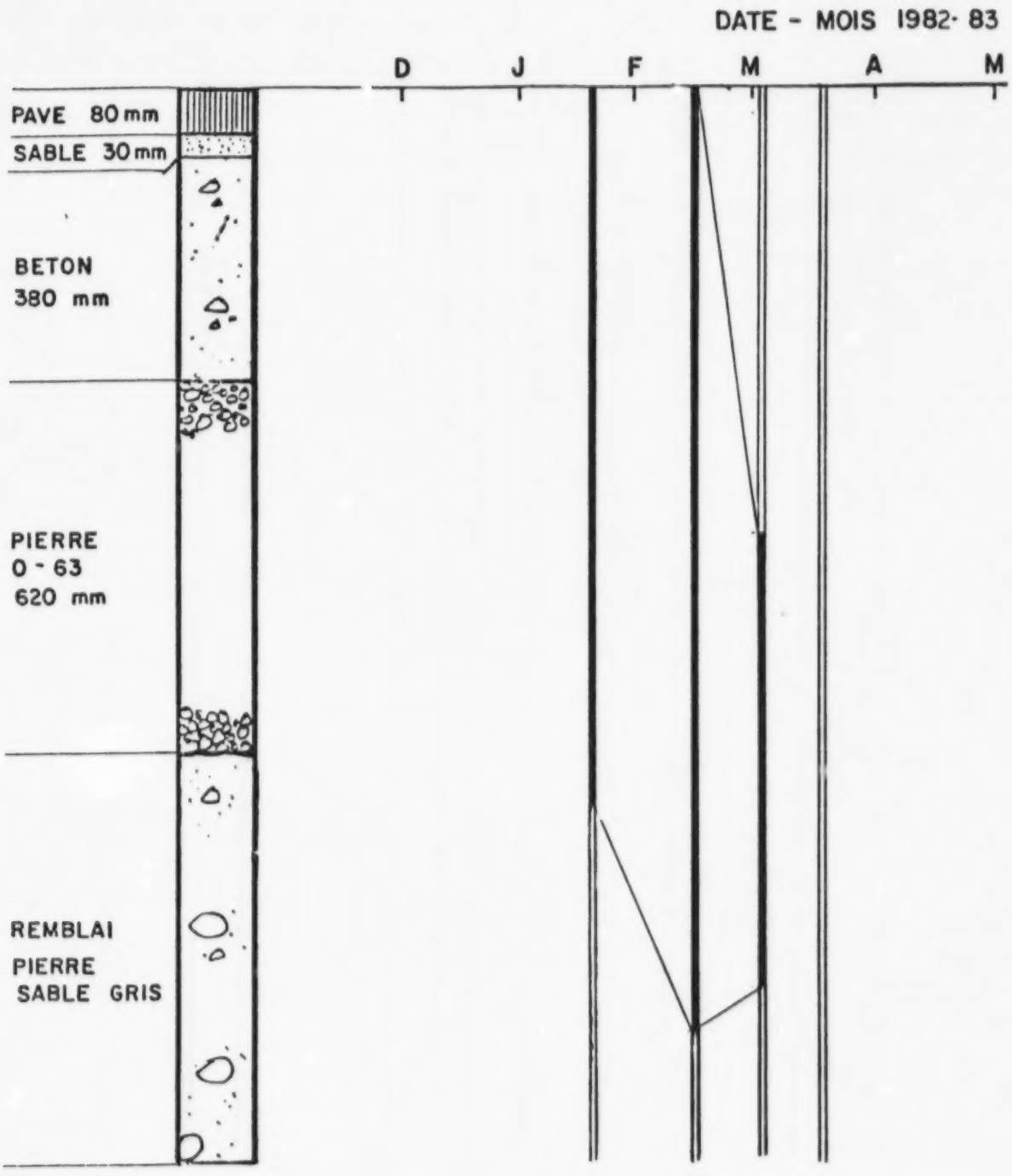
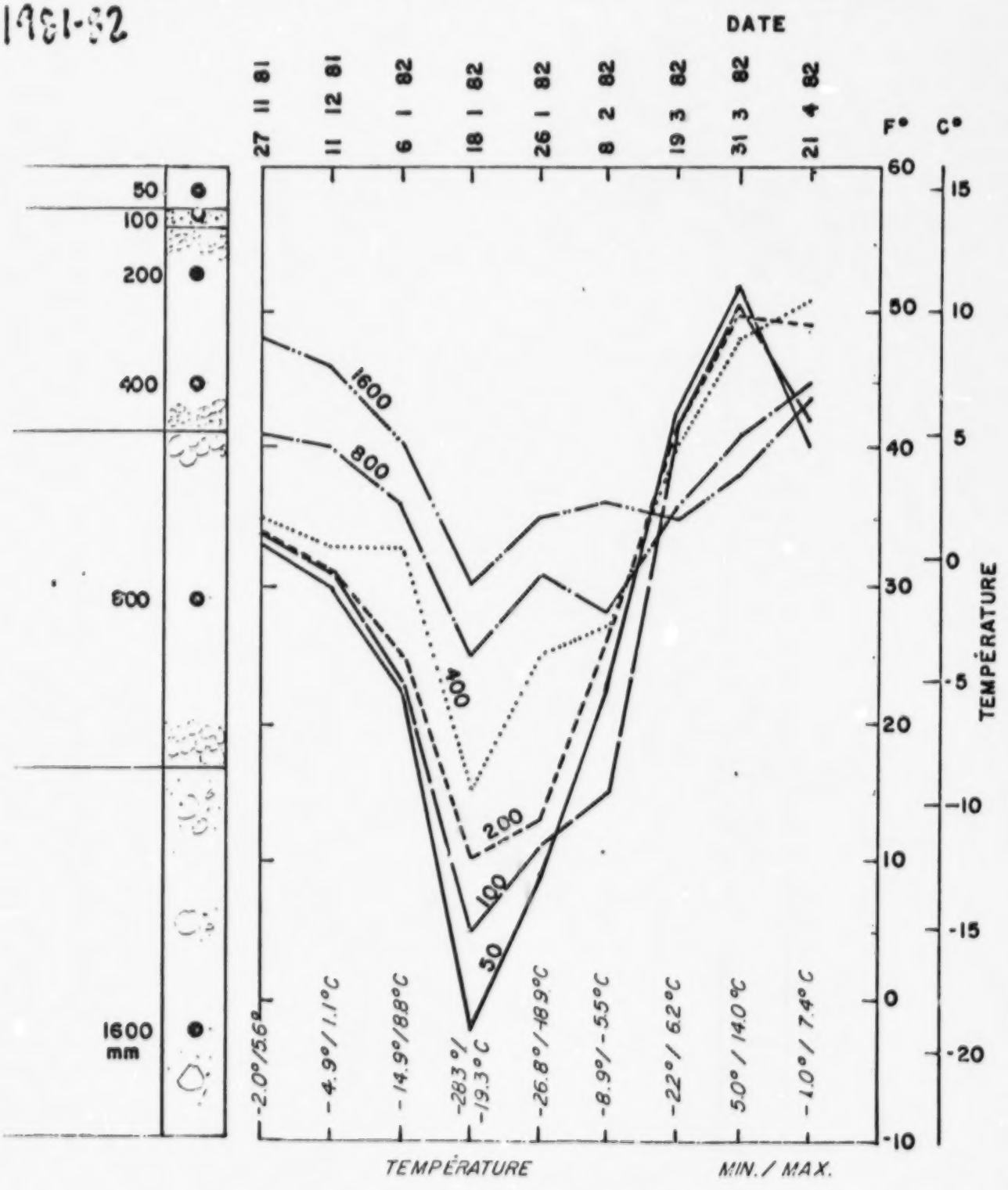


VARIATION DE TEMPÉRATURE

DALLE D

1981-82



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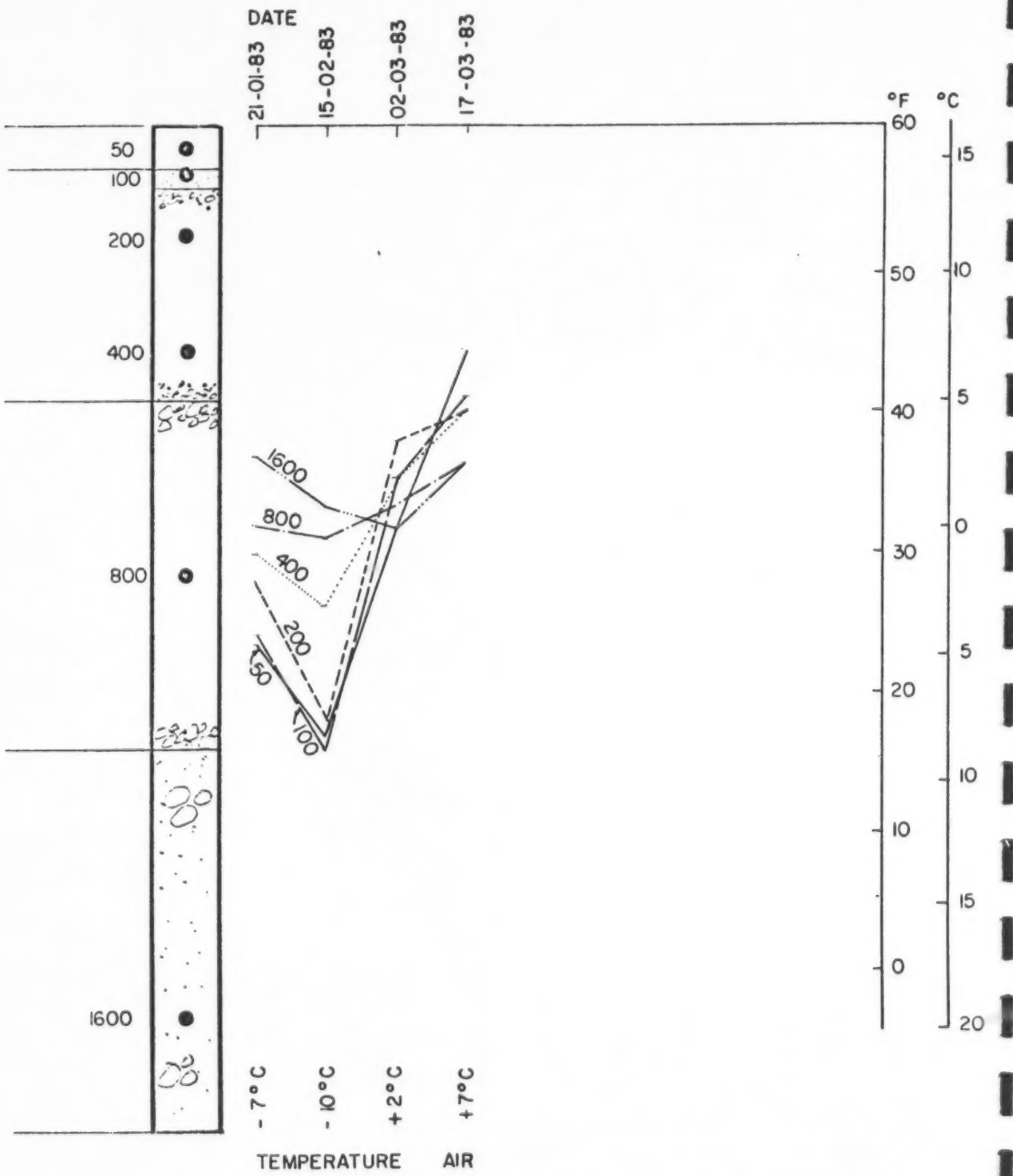

VARIATION DE L'HUMIDITÉ

DALLE D

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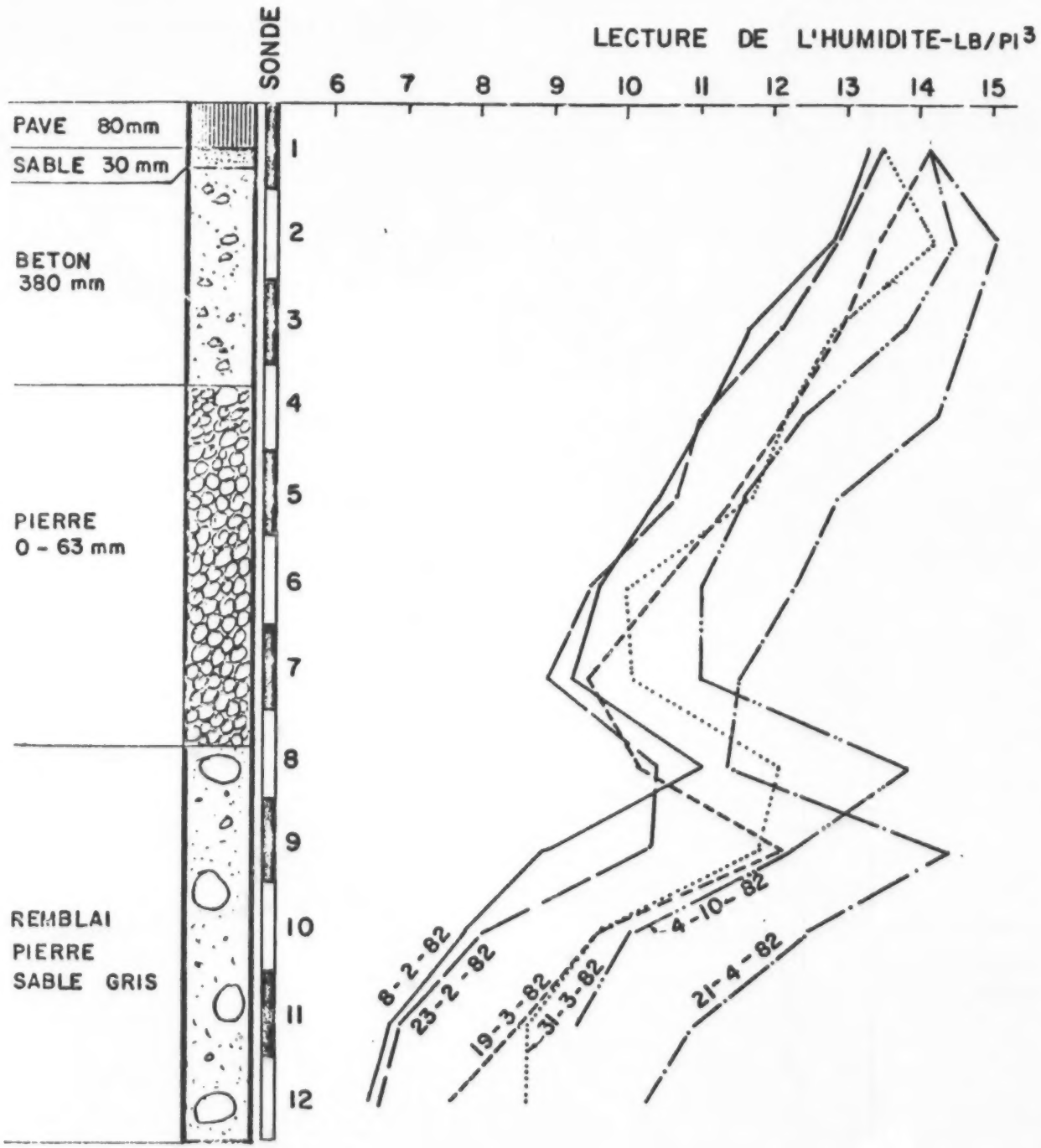

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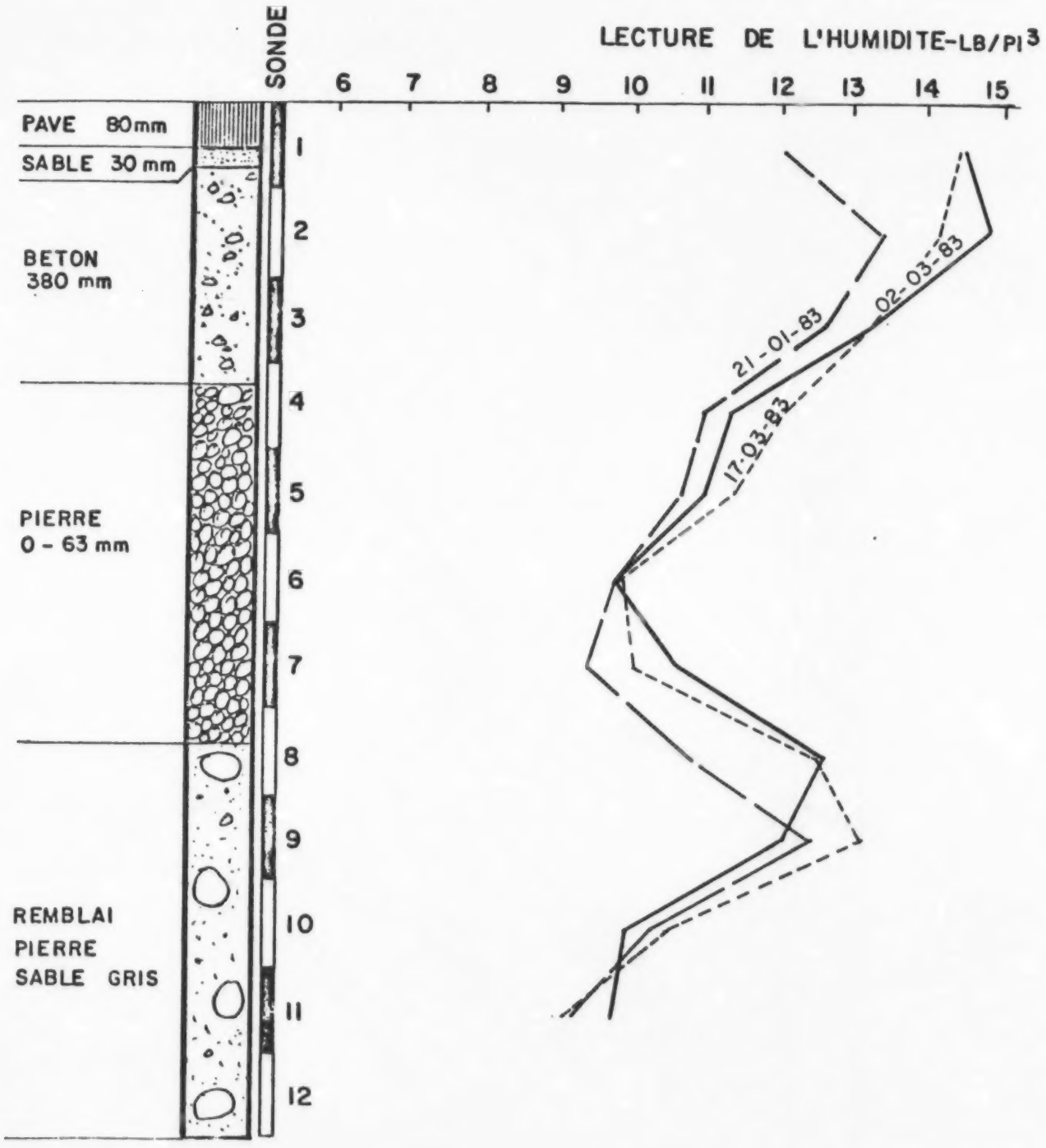

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